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SOCIETY AND RELATED ACTIVITIES

RESULTS ACCOMPLISHED BY A PROGRESSIVE SISTER SOCIETY

By **J. F. Kelly***

(Reprinted from July 1932, Iron and Steel Engineer.)

The AMERICAN WELDING SOCIETY, one of the leading and one of the most progressive national engineering societies in the United States, will soon be enjoying its fourteenth anniversary.

It started with a membership of about one hundred and has grown to over fifteen hundred members, many of them being the leading technical and practical welding authorities of the United States.

The AMERICAN WELDING SOCIETY is today the fountain head of all welding activities and its specifications and codes are not only the recognized standards but they represent the combined efforts of the welding engineering fraternity in this country.

For example, in the early days a wide variety of nomenclature and definitions was in vogue. Welding experts did not talk the same language. To meet this need a committee was appointed to draw up standard nomenclature, definitions and symbols. Not only has the matter of the common language been successfully solved, but standard symbols enable the draftsman to specify clearly and concisely the type, location and size of welding to be used, thereby eliminating confusion and expense. Other standards developed by committees of this Society or its research department known as the American Bureau of Welding, include standard tests for welds, standards for arc welding apparatus, specifications for welding wire and gages for measuring dimensions of welds.

Perhaps the growth of welding during the past few years has been, to a large extent, due to the maintenance of quality standards and procedure specifications. In this way it is possible to eliminate the doubt as to the quality of workmanship, and engineers have, in general, come to look upon welding with greater confidence.

In the matter of codes the Society has also taken a leading and active part. The first code issued by the American Society of Mechanical Engineers for pressure vessels included some very restrictive measures against welding. The Society through its research department carried out an exhaustive series of tests which furnished authentic data to back up the Society in its demands for a more lenient code which would be fair to welding.

These demands led to the appointment of suitable joint committees which, through research work and deliberations, have resulted in the providing of a new code which permits the use of welding to an extent considered impossible a few years ago by even the most enthusiastic welding engineer.

In the structural welding field the Society has taken the initiative and has issued a building code which has been adopted by over a hundred cities and municipalities. The values have been used in the design and erection of several hundred important welded structures.

The research work of the Society is conducted under the auspices of a separate board, known as the American Bureau of Welding. This board is also affiliated very closely with the National Research Council and has representation from some fifteen leading scientific and engineering organizations.

An outstanding report of the American Bureau of Welding was recently issued. It is the report of the Structural Steel Welding Committee. It is also indicative of cooperation on a wide scale. The program as carried out and reported included the welding and testing of nearly 2500 specimens.

The participants in the investigation included three steel mills, 39 fabricating shops, 61 welders, 18 inspectors and 24 testing laboratories. The work was distributed over the central and eastern portions of the United States. A Canadian laboratory provided pilot or preliminary information based on the testing of 342 specimens welded in three Canadian shops. About five years have been spent on entire investigation. This report, occuping more than 200 printed pages, is available.

Some of the aims of the Society may be briefly summarized as follows:

1. To collect and make available authentic and up-to-date information on welding, gas and electric arc cutting and to be recognized as the authoritative source of such information by manufacturers in the welding field, users of the process, engineering societies and legislative bodies.

2. To provide a means for the interchange of knowledge and experience to aid in the solution of the problems of the welding art—technical, ethical and commercial—through cooperative effort.

3. To be the agent of the industry in cooperative research on important technical problems and to serve as a common spokesman of the industry in matters pertaining to its welfare.

4. To provide opportunities for social intercourse, and thus to promote a better understanding among manufacturers, users and scientists in the welding field, and to foster a spirit of cooperation for the common good.

5. To encourage the development of welding through improvements in present applications and expansion of its uses into new fields, and to develop technical and ethical standards for the welding industry.

The steel industry, as a whole, could with profit follow the work and activities of the AMERICAN WELDING SOCIETY. The steel industry, in its endeavor to find new uses, new markets, new outlets for steel, should lend its moral as well as its financial support to the AMERICAN WELDING SOCIETY, which devotes three hundred and sixty-five days of each year to developments in welding. The results accomplished by this Society's activities will, in the future, pay a handsome dividend to the steel industry when taking into consideration that with the use of welding, fabricated rolled steel products have a potential market of twenty million tons of steel annually.

The Welding Engineering Division of the Association of Iron & Steel Electrical Engineers will cooperate in the future with the AMERICAN WELDING SOCIETY to the fullest extent. A close cooperation between these two Societies will, without doubt, prove a distinct advantage to the steel industry in its campaign for new uses, new markets, new outlets for steel.

The Association of Iron & Steel Electrical Engineers, composed of twenty-two hundred members, desires at this time to express its appreciation to the AMERICAN WELDING SOCIETY on its fourteen years of progressive and successful efforts toward solving the problems of welding, especially for its splendid work on behalf of the iron and steel industry. Its past certainly assures a successful future.

* Mr. Kelley is Managing Director of A. I. & S. E. E.

Obituary

Jesse W. Gray met with an accident which resulted in his death while returning one evening from a visit to the city of Altoona. The rear tire of his automobile blew out, resulting in a collision with the curb at a stone culvert. The car turned over and at the hospital it was discovered that Mr. Gray suffered from internal injuries and concussion. He died on Saturday morning, July 8th at the Tyrone Sanitarium, Tyrone, Pa. Mr. Gray was born on December 14, 1904 and after leaving Penn State College he took a course at the Lock Haven State Normal School and taught public school for a period of three years before joining with the Westinghouse Electric & Manufacturing Company where he specialized in arc welding.

Mr. Gray has been very active in the affairs of the Society since his location in New York, having served a number of times on the National Dinner Committee. He was also an active member of the Board of Directors of the New York Section of the Society and of the National Membership Committee.

TENTATIVE PROGRAM
THIRTEENTH FALL MEETING
AMERICAN WELDING SOCIETY
OCTOBER 2-6, 1933

Book-Cadillac Hotel
Exposition–Convention Hall
Detroit, Mich.

Important Notice

Members and guests are requested to register immediately upon arrival, Book-Cadillac Hotel, as admission to Exposition and participation in social events and inspection trips will be by badge only.

Upon registering you will be furnished with copy of the Journal containing papers to be presented at the technical sessions. The supply of Journals is limited, one to each registrant, and on this account you are urged to retain your copy throughout the meeting.

All technical sessions and committee meetings will be held at the Book-Cadillac Hotel.

All sessions will start promptly as scheduled.

There will be no stenotype reporter. Members of the Society desiring to discuss papers are urgently requested to prepare their discussion in writing in advance of the meeting and to send copies to headquarters as those preparing written discussion will be given preference at the sessions. Members and guests giving extemporaneous discussion at meetings should forward a written transcript of their discussion as soon after the meeting as possible.

Monday, October 2nd

Morning

REGISTRATION

Facilities will be provided throughout the week from 9:30 A.M. to 5:00 P.M., commencing Monday, October 2nd.

Afternoon

OPENING SESSION

1:45 P.M. Presiding Officer—M. P. Bailey, Chairman, Detroit Section.

Address of welcome by Mayor and response.

TECHNICAL SESSION

2:15 P.M. Presiding Officer—Frank P. McKibben, President.

A paper on "Gas Cutting of Structural Steel." (Author to be announced later.)

"Impact Resistance of Welded Joints—A Survey of the Literature," by W. Spraragen.

"Use of Shielded Carbon Arc in Class I Welding," by E. W. P. Smith, Consulting Engineer, The Lincoln Electric Co.

"Impact Values of Weld Metal," by J. C. Hodge, The Babcock & Wilcox Company.

Evening

6:30 P.M. Dinner meeting, Board of Directors, Book-Cadillac Hotel.

Tuesday, October 3rd

Morning

TECHNICAL SESSION

9:45 A.M. Presiding Officer—J. J. Crowe, Senior Vice-President.

"Manufacture of Small Everdur Tanks by Welding," by Ira T. Hook, American Brass Co.

"Welding of Chrome Alloy Steels," by W. B. Miller, Union Carbide & Carbon Res. Labs.

"Gas Welding of Aluminum." (Author to be announced later.)

"Welding of Non-Ferrous Metals and Alloy Steel," by representative, Thomson-Gibb Co.

"The Welding of Inconel," by J. G. Schoener & F. G. Flocke, The International Nickel Company, Inc.

Afternoon

TECHNICAL SESSION

2:00 P.M. Presiding Officer—C. A. Adams, Director, American Bureau of Welding.

FUNDAMENTAL RESEARCH IN WELDING

"Physical Properties of Welded Cast Steel," by Chas. H. Jennings, Westinghouse Electric & Manufacturing Co.

"Tensile Tests of Welded and Riveted Structural Members," by R. P. Davis, Dean, and G. P. Boomsliter, West Virginia University.

"Electric Arc Welding under Water," by N. S. Hibshman and C. D. Jensen, Asst. Profs., Electrical and Civil Eng., Lehigh University, and W. R. Harvey, Asst. Prof., Lehigh University.

"The Effects of Motor Reactions in Electric Welding," by W. R. Woolrich, Prof., Mech. Eng., J. Tarboux, Prof. Elec. Eng., University of Tennessee, and C. T. Raymo and W. B. Parker.

"Fatigue of Metals," by G. E. Thornton, Dept. of Mech. Eng., The State College of Washington.

"Internal Stresses in Welds," by C. T. Schwarze, Dept. of Civil Eng., New York University.

"Magnetic Characteristics of Deposited Metal," by Wendell F. Hess, Asst. Prof. of Elec. Eng. & Physics, Rensselaer Polytechnic Institute.

Evening

7:30 P.M. Book-Cadillac Hotel. Conference and meeting of Fundamental Research Committee, American Bureau of Welding. H. M. Hobart, Chairman, presiding.

This conference is scheduled for the benefit of university research workers in the fundamentals of welding.

Wednesday, October 4th

Morning

TECHNICAL SESSION

9:45 A.M. Presiding Officer. R. E. Powell, Western Electric Co.

RESISTANCE WELDING SESSION

"Automatic Controls for Electric Welding," by H. W. Roth, President, Controloweld, Inc.

"Controlled Welding," by O. C. Frederick, General Electric Company.

"Precision Spot Welding with Tube-Controlled Contactors," by C. Stansbury, Engineering Dept., Cutler Hammer, Inc.

"Timing of Spot Welders with Reference to Current Flow," by D. C. Wright, Chief Eng., The Elec. Controller & Mfg. Co.

Afternoon

INSPECTION TRIP

1:15 P.M. Leaving Book-Cadillac Hotel.

This inspection trip, arranged by the Detroit Section, will include visits to the welding departments of the Kelsey Hayes Wheel Corporation, the Ford Motor Co. and an automobile body plant.

Thursday, October 5th

Morning

TECHNICAL SESSION

9:45 A.M. Presiding Officer. (To be announced later.)

"Shipbuilding Welding," by G. H. Moore, Jr., Newport News Shipbuilding & Dry Dock Co.

"Welding of Structural Nickel Steel," by W. L. Warner, Watertown Arsenal.

"Welding of Galvanized Steel," by Leon C. Bibber, Bureau of Construction & Repair, Navy Dept.

"An Aircraft Manufacturer's Experience with Welding Quality Control," by T. H. Speller, Welding Engineer, and P. N. Jansen, Factory Manager, Curtiss Aeroplane & Motor Co., Inc.

Afternoon

TECHNICAL SESSION

2:00 P.M. Presiding Officer—M. P. Bailey, Chairman, Detroit Section.

"Shop Setup for Pre-fabrication of Welded Machinery," by Mr. Kondal, Wickes Boiler Co.

"Modern Rail Welding without Interrupting Traffic," by J. H. Deppeler, Metal & Thermit Corp.

"Pressure Regulators and Regulation Problems," by George M. Deming, Apparatus Research & Development Dept., Air Reduction Sales Co.

"Welding in the Automotive Industry." Author to be announced later.

"Automobile Welding." Author to be announced later.

Evening

7:00 P. M. Annual Banquet followed by dancing. Book-Cadillac Hotel.

The Practical Design of Welded Steel Structures*

By H. M. PRIEST

✦ Paper presented at the March 23, 1933 meeting of the New York Section, A. W. S., by H. M. Priest, Designing Engineer, American Bridge Company

Introduction

THE literature of structural welding is already too extensive to be fully reviewed within the limits of a single article. It has been necessary, therefore, to abstract whatever seemed of outstanding value in numerous published articles, pamphlets, bulletins and reports, from which the writer has drawn much of his material.

It will be the aim of this article to present the essentials of the design of welded steel construction, with a sufficient background of collateral information that the reader may have a somewhat clearer understanding of structural welding.

To the engineer or contractor in search of information on welding the question naturally comes, "Where is such information to be found?" The field is too broad to mention all the sources but a few representative ones are listed below:

1. Publications of the AMERICAN WELDING SOCIETY:
   - (*a*) Journal—published monthly.
   - (*b*) Report of the Structural Steel Welding Committee.
   - (*c*) Code for Fusion Welding and Gas Cutting in Building Construction, Edition 1930.
   - (*d*) Bulletin on Nomenclature, Definitions and Symbols.
   - (*e*) Specification for Welding Rods and Electrodes.
2. Magazines:
   - (*a*) *Welding.*
   - (*b*) *The Welding Engineer.*
   - (*c*) *Engineering News-Record.*
   - (*d*) *Steel.*
   - (*e*) *Industry and Welding.*
   - (*f*) *Iron Age.*
   - (*g*) *The Welding Industry* (London, England).
3. Manufacturers' Bulletins:
   - (*a*) General Electric Company.
   - (*b*) Lincoln Electric Company.
   - (*c*) Westinghouse Electric and Manufacturing Co.
   - (*d*) Air Reduction Company.
   - (*e*) The Linde Air Products Company.
4. Reports:
   - (*a*) U. S. Bureau of Standards.
   - (*b*) University Laboratories.
5. Proceedings of Technical Societies:
   - (*a*) American Institute of Electrical Engineers

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CHART CLASSIFYING THE PRINCIPAL WELDING PROCESSES

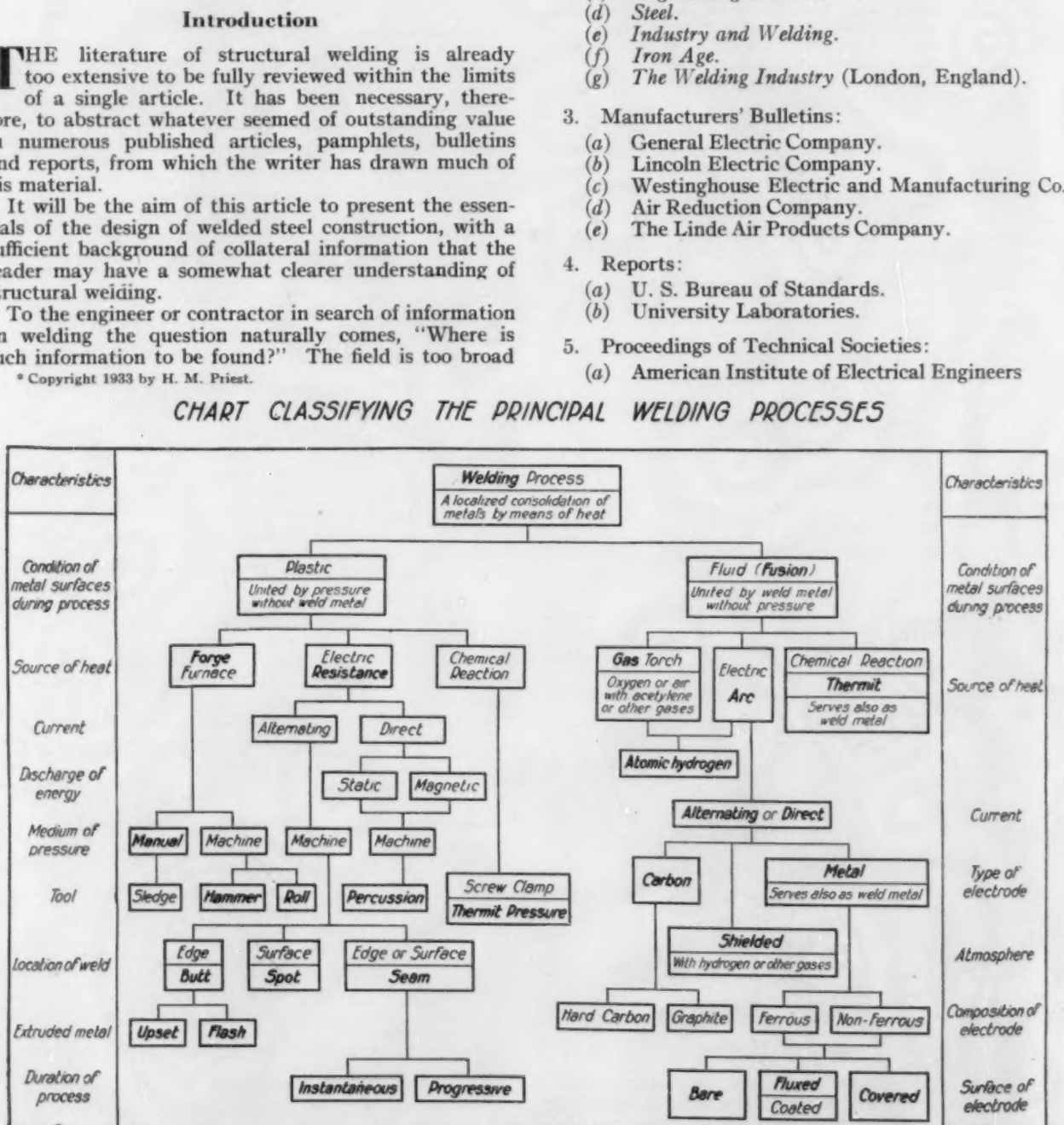


Fig. 1

(*b*) American Society of Mechanical Engineers.
(*c*) American Society for Testing Materials.

Welding Processes

What is welding? The approved definition is "a localized consolidation of metals by means of heat." Mr. F. T. Llewellyn published an excellent chart classifying the principal welding processes, which is reproduced in Fig. 1. Most structural welding is done by two of the main fusion processes, with electric arc welding predominating. Buildings have been fabricated using both arc and oxyacetylene welding. One of the plastic processes, namely, electric resistance welding is extensively utilized in the manufacture of steel joists and light trusses of skeleton form.

It is not within the scope of this article to discuss at length the technique of the welding operations, but rather to present only a general picture which will add to an intelligent appreciation of the subsequent remarks.

Metal Arc Welding

The welding heat is obtained from the electric arc formed between the electrode and the parts to be welded. Both direct and alternating current are employed, with the former most widely used. Figure 2*A* represents the direct current process in which a generator, driven by electric motor or gasoline engine, supplies the current. Investigators have found that more heat is liberated on the positive side of the arc than on the negative. Hence the usual practice of attaching the positive terminal of the generator to the base metal (parts to be welded) which is of greater extent than the wire and requires more heat to bring it up to the welding temperature. Under certain conditions, as when welding thin material or using some of the covered electrodes, the connections are reversed, giving rise to the expression, "welding with reversed polarity."

In the process of arc welding the electrode melts and unites with the base metal to form the weld. A distinctive feature of arc welds is the crater formed immediately under the electrode.

Oxyacetylene (Gas) Welding

Figure 2*B* depicts the essentials of the gas-welding process in which the welding heat is derived from the combustion of acetylene and oxygen at the tip of a blowpipe or torch. The torch can be fitted with interchangeable tips with varying sizes of orifices, depending upon the amount of heat required. Suitable gages and regulators permit of the control of the gas pressures to obtain the best results. The base metal is brought up to the proper welding temperature by directing the flame upon it, after which the welding rod is introduced into the flame and is melted to form the weld.

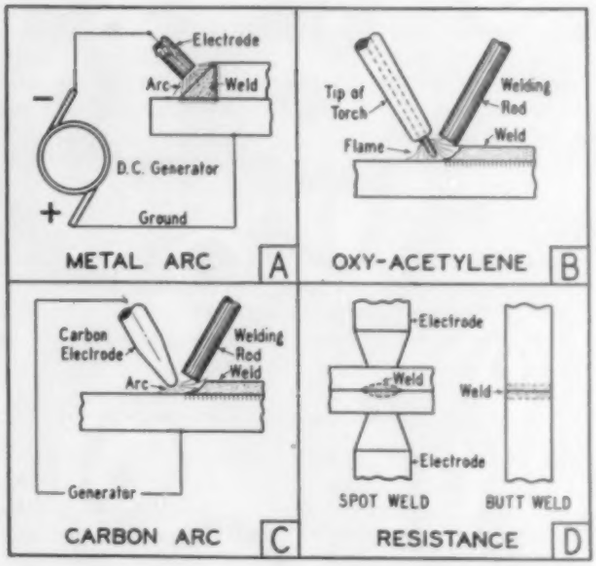
Carbon Arc Welding

Carbon arc welding, Fig. 2*C*, is similar to gas welding, in that a welding rod is introduced into the arc formed between the carbon electrode and the base metal. In some cases, when welding thin materials, the heat of the arc is used to melt the parts and fuse them together without the addition of any filler material from a welding rod.

Resistance Welding

The principle of resistance welding, Fig. 2*D*, consists in passing an alternating current of high amperage and low voltage through metals while under pressure and using the heat produced by the electrical resistance at

Fig. 2



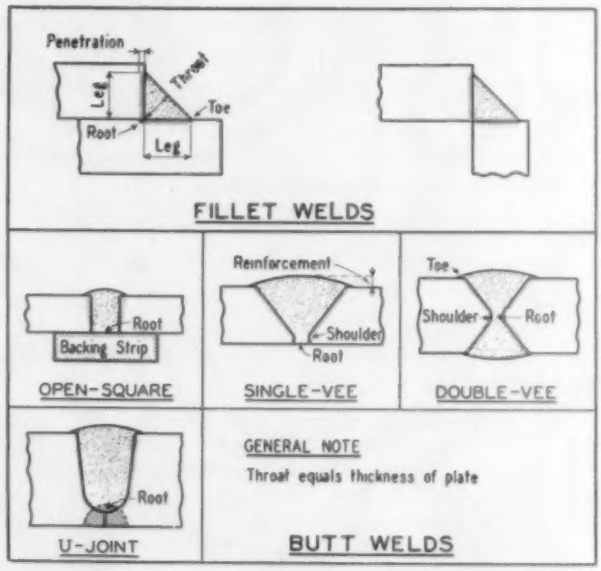


Fig. 3

the point of contact to bring the material to a plastic state. The metals are welded in somewhat the same way as in forge welds.

Welding Rods and Electrodes

There are many rods and electrodes on the market from which the user may select the type best suited to his particular requirements. In structural work it has hitherto been the common practice to weld with high strength rods for the gas process and bare-wire electrodes with the arc process.

Electrodes are commonly divided into three classes:

1. Bare wire.
2. Lightly coated wire.
3. Heavily coated wire.

The first class has been widely used in this country with satisfactory results, while European practice has more generally adopted the heavily coated electrode. There is, however, an increasing use of the coated wires

in the United States, partly because of superior characteristics to be obtained with these wires and an increased rate of weld deposition which they make possible. It may not be amiss to say that with increased speed of welding there is need of care in seeing that the speed does not exceed that at which sound welds and thorough penetration are obtained.

The manufacturers of electrodes quite generally state the voltage and current values to be used with their product. These vary with the size of the wire and the thickness of the material to be welded. The smaller the wire or the thinner the material, the less the current should be for the proper results. Too heavy current is apt to burn the steel and to produce porosity in the welds. It is, therefore, wise to be guided by the advice of responsible manufacturers.

Form of Welds

The two fundamental forms of welds are those shown in Fig. 3 and designated by the terms fillet and butt welds. The fillet form is used for all lap joints and the butt form when the parts to be welded are in alignment. The examples of butt weld joints are a few of the more typical ones in common use. Welding literature is so full of certain terms relating to welds that it may not be out of place to briefly review their definitions.

Base Metal.—The parent material to be welded or cut.

Filler Metal.—Material specially prepared for addition to the weld in some forms of the fusion welding processes (welding rod, electrode and thermit mixture).

Weld Metal.—The material composing the weld.

Bead.—A single linear continuous deposit of weld metal made by the gas- and arc-welding processes.

Weld Penetration.—A dimensional expression of the depth of the fusion zone below the original surface and/or edge planes of the base metal.

Weld Size.—A dimensional expression of the cross-sectional designed value of the weld. The size of a fillet weld, made by the gas- and arc-welding processes, is the designed length of its legs, and the size of a butt weld, made by the gas and arc processes, is its net or unreinforced throat dimension in inches.

Weld Length.—A dimensional expression of the unbroken length of the weld. For an arc weld this length is that of the full cross section of the weld, exclusive of the length of any craters.

Weld Throat.—The minimum thickness of a weld along a straight line passing through the root. As interpreted in the Code for Fusion Welding and Gas Cutting in Building Construction, the throat of a fillet weld shall be the distance along a line from the root to the hypotenuse at right angles thereto of the largest isosceles right triangle that can be constructed in the cross section of the fillet weld; and the throat of a butt weld shall be equal to the thickness of the thinner part joined.

Gas Cutting

Closely allied to welding is its counterpart, gas cutting—the severing of ferrous metals by means of the chemical behavior of oxygen in the presence of ferrous metals at high temperatures.

The AMERICAN WELDING SOCIETY'S Building Code contains a section on "Gas Cutting" from which these clauses are quoted:

(*a*) "Gas-cut edges shall be smooth and regular in contour."

(*b*) "Gas cutting may be used in the preparation of base metal parts for welding, provided the edges so cut are thoroughly cleaned after cutting so as to expose clean steel."

As a refinement of hand-guided cutting it is of interest to know of the accuracy which can be obtained when the torch is mechanically guided, as in a lathe. The writer has seen a block of steel $1^1/_4$ in. thick which was flame-cut within a tolerance of 0.002 in. and the line of cut bisected small prick punch marks.

A recent development, known as flame machining, uses the oxyacetylene cutting torch at a flat angle to the surface of the metal so that the cut does not extend entirely through the piece. By varying the slope of the tip and the gas pressures it is possible to make cuts of special shapes. This process promises to be of great assistance in the economical preparation of edges of material that is to be butt welded.

Qualification Tests for Welders

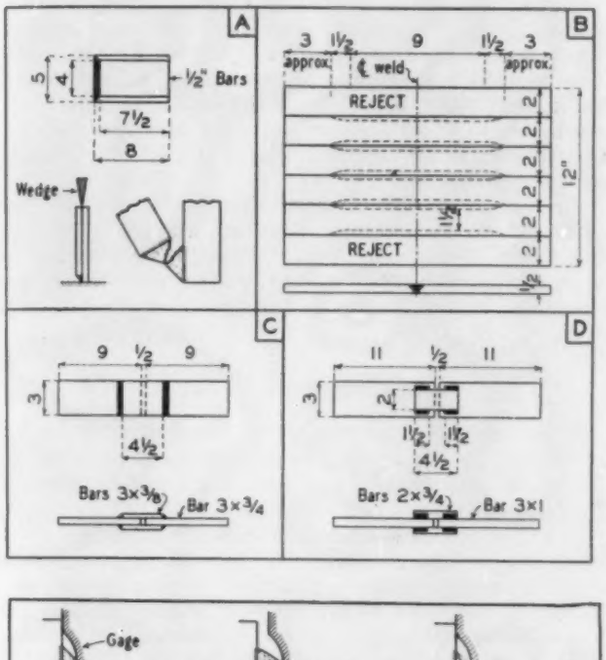
It is generally recognized that every welder should be required to demonstrate his fitness and ability before being permitted to weld on the finished product. Various "qualification tests" have been devised according to the line of work in which the welding was to be done. Figure 4 illustrates four forms of test specimens which have been used in qualifying welders for structural steel.

Specimens *A* and *B* were used by the Structural Steel Welding Committee for testing all welders who took part in its investigation. The so-called wedge test, *A*, offered a rapid and inexpensive method of securing preliminary information as to a welder's general ability. After the weld was completed and had cooled, the specimen was torn apart by driving a wedge between the bars and prying them apart until the weld ruptured. Inspection of the fractured metal then showed the quality of the deposited metal, the fusion of the deposited metal with the base metal and the degree of penetration at the root of the weld. A welder had to pass this test before being allowed to proceed with the remainder of the test.

The butt-welded form of Specimen *B* was the most important test and provided definite information in regard to the physical strength of the welds. Three specimens were welded in a flat position and two in a vertical position. The standard A. S. T. M. coupons were cut out and machined to the required size. Since the original requirements of strength were different for gas and arc welding it will perhaps be better if the final recommended requirements be given as the two sets of values were brought into agreement. Instead of a total of five specimens the Committee recommended two, one to be welded in a flat position and the other in a vertical position. The eight coupons should show a minimum ultimate strength per sq. in. of throat area of forty-five (45) kips with an average of fifty-two (52) kips. (A kip is 1000 lb.)

This type of specimen is not particularly suited to structural work. In the first place it is for butt welds, where the general practice of structural fabrication uses fillet welds, and it is felt that any test specimen should conform more nearly to practice. A second objection is that of cost. There is a very considerable amount of machining required to produce the coupons and the extra labor and expense become a very appreciable amount. However, in work requiring butt welding, it is an excellent qualification test.

Fig. 4



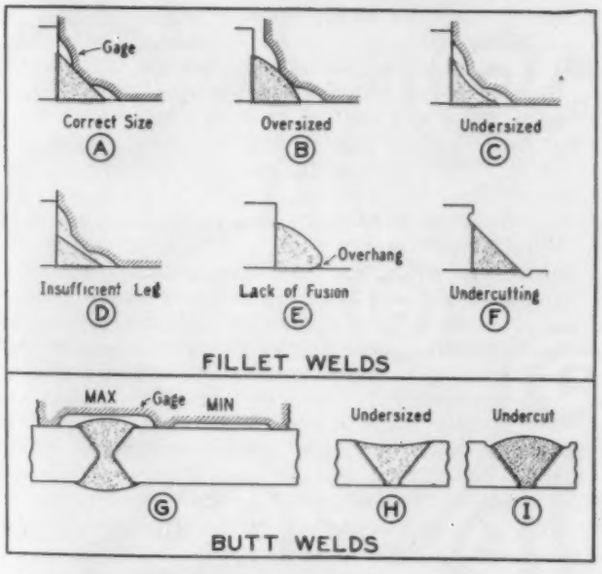


Fig. 5

The Committee took cognizance of these facts and recommended the two types of fillet-welded specimens, *C* and *D*.

The sizes adopted were selected so that the specimens would come within the dimensional and load limits of the testing machines to be found most generally in the smaller laboratories, namely, 100,000 lb. ultimate capacity.

It was recommended that two specimens be welded in each of the positions (flat, vertical and overhead) that will be required in the welder's subsequent work. When tested in tension, each specimen should show a minimum ultimate strength of 12 kips per lin. in. of weld, equivalent to a total load on the specimen of 72 kips.

Specimen *C* with end-fillet welds has much to commend its general use. Its advantages over Specimen *D* might be enumerated as follows:

1. The bars can be readily obtained from stock material.
2. The welds cannot overrun on the vertical leg or in length.
3. Since the bars are of the same width it facilitates the asembling and lining up of the specimen.
4. With half as many welds, there are half as many points of starting and stopping.

It is decidedly to the interest of all concerned that a uniform qualification test be established throughout the structural steel fabricating industry. For a preliminary check, when necessary, Specimen *A* will serve the purpose adequately. For the tension test, Specimen *C* is excellent: two specimens to be welded in a flat position, two vertical and, in case overhead welding is to be done in subsequent work, two specimens shall be required in this position. Ordinarily no shop welds will be made in an overhead position but for field welders it might be well to require them to demonstrate their ability to make welds in this position. The Committee's requirement for strength of 12 kips per lin. in. of weld is satisfactory.

Inspection

With the widening uses of welding in many lines of industry has come the necessity for inspection to insure the quality of the finished welds. As yet no method of testing, comparable to "tapping rivets," has been devised although several have been put forward which are partially successful. Most of them work best with butt welds and are, therefore, not as applicable to the fillet welds more commonly used in structural practice.

A few of the prominent methods will be very briefly outlined. The most effective and also the most costly are the X-ray and the Gamma-ray methods by which photographs of the welds are taken, utilizing the penetrating radiations of the X-ray tube and of radium, respectively. There can be no doubt of the efficacy of either method in locating the character and extent of any defects. For Class 1 unfired pressure vessels the A. S. M. E. Boiler Code makes the X-ray examination of welds one of the requirements of their fabrication.

Two other methods depend upon the variations in a magnetic circuit due to imperfections when a magnetic flux is passed through the weld. With the "magnetographic" method a piece of paper is placed over the weld upon which are sprinkled iron filings. The filings collect at points where gas pockets, slag inclusions and poor penetration occur. Experience will enable an inspector to interpret the patterns quite accurately. The "bridge" method uses the Wheatstone bridge and a galvanometer to read the variations in the circuit as the exploring device is moved along the weld.

Still another method utilizes the stethoscope and depends for its successful use upon the ability of the inspector to detect variations in the sounds produced when the weld is tapped lightly with a hammer along its length. The very simplicity of the apparatus gives this method an immediate appeal and it has found considerable use in certain fields.

For structural welding the visual inspection by a competent inspector during and/or upon completion of the welding is satisfactory, but in addition there are several very definite things which can and should be done to insure workmanship of good quality.

First, none but properly qualified welders should be permitted to do structural welding, or any welding where strength is a requisite consideration. The investigation of the Structural Steel Welding Committee showed that, in general, the higher a welder's qualification rating, the higher was his subsequent performance on the program test specimens.

Second, a definite procedure control should be followed. This involves the preparation of the parts to be welded, the selection of the proper electrodes or welding rods, the technique of the welding process and provision for adequate supervision. Reputable manufacturers of welding equipment and supplies are naturally vitally interested in the successful application of their products and many of them maintain large service organizations to which the user may turn for information and guidance.

These two steps will go a long way toward the production of sound welds of the required strength. There is left the inspection of the finished weld. Observation of a welder at work will furnish information as to the general character of his welds. If arc welding, he should hold a short arc and be able to maintain it without frequent interruptions. It will also be possible to judge the amount of penetration which he is obtaining. The meters on the welding machine can be read to see that the operator is using the proper current and voltage. Then one should note whether the joint surfaces are clean and free from dirt, paint or grease which interfere with the welding process. Of course, the inspector cannot stand over the welder continuously but an occasional observation is sufficient.

Visual inspection will yield information to the trained eye of the engineer or inspector. A number of essential points are shown in Fig. 5. The welds should be gaged for size as shown in Sketches *A*, *B*, *C*, *D* and *G*. The dotted line in the fillet welds indicates the correct or design size. Butt welds are reinforced and the additional weld metal must have a minimum height and not exceed a certain maximum value, depending upon the size of the weld. The gages used by the Structural Steel Welding Committee are fully described in the report under Section 2-5-9.

The surface of the weld should be fairly smooth with regular and even ripples caused by the steady advance of the electrodes or welding rods. The edges of the weld should appear to thoroughly merge with the base metal. Sketch *E* indicates a form of weld which may occur when the operator fails to secure full penetration, with a resulting overhang. Often the point of a knife or the edge of a sharp chisel can be inserted under the weld. Drops of metal quite generally spattered on the base metal indicate that too long an arc was probably held with the consequent likelihood of porosity and inferior penetration. The use of too much heat (i.e., current) is sometimes indicated by the welds turning a reddish color when they have cooled. Sketch *F* shows a common form of weld when too much heat has been used in order to get speed in welding. In some cases, as in Sketch *I*, the undercutting might be a serious defect due to the reduction in cross-sectional area of the member, and furthermore it acts much as a nick around which stresses always concentrate.

No amount of printed instructions can take the place of actual experience with practical welding operations. One has to see welding done, both in the shop and in the field, and examine the finished product in order to have a working understanding of the process and its application.

Ultimate Weld Failures

It is valuable information to know how welds fail when they are stressed to their ultimate carrying capacity. A few examples of typical cases will suffice to illustrate the usual manner of failure.

In Fig. 6 are shown several types of welded test specimens which can be imagined as being given tension tests to produce failure. Sketch *A* is representative of the welds subjected to longitudinal shear. Failure will invariably occur through the critical throat section unless there is an extensive lack of fusion on one of the legs of the fillet, in which case failure may take place between the base metal and the weld. End welds under transverse shear also fail through the throat, as indicated in Sketch *B*. There is a condition with end welds, as shown in Sketch *C*, in which the eccentricity of the bars causes a moment on the weld that forces failure to occur along the vertical leg of the fillet.

Butt-welded specimens behave in two ways depending upon the quality of weld metal and reinforcement. If the reinforcement is of sufficient amount and is left on the specimen, failure will probably occur in the base metal at some distance from the weld. Should the reinforcement be removed by milling or grinding, then failure will occur through the throat of the weld. It must be remarked that this statement is based on the use of bare electrodes in making the welds. With high strength rods and electrodes it is usual to produce welds of greater strength than the base metal of ordinary structural steel.

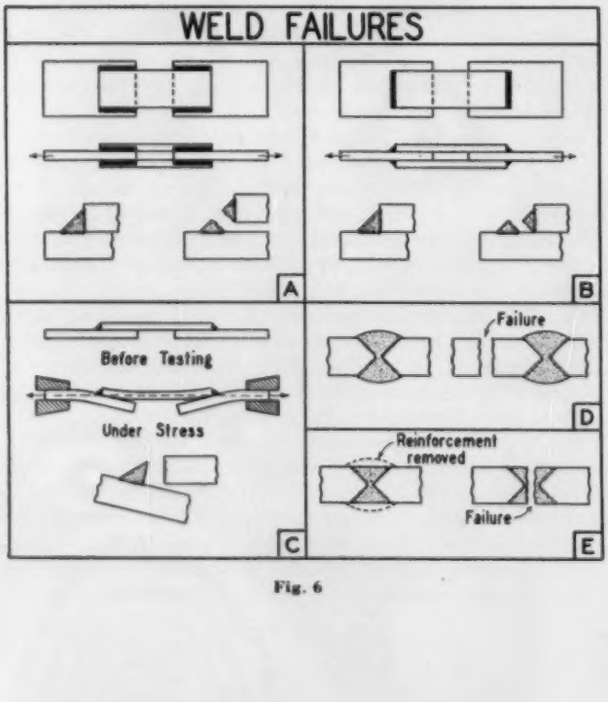


Fig. 6

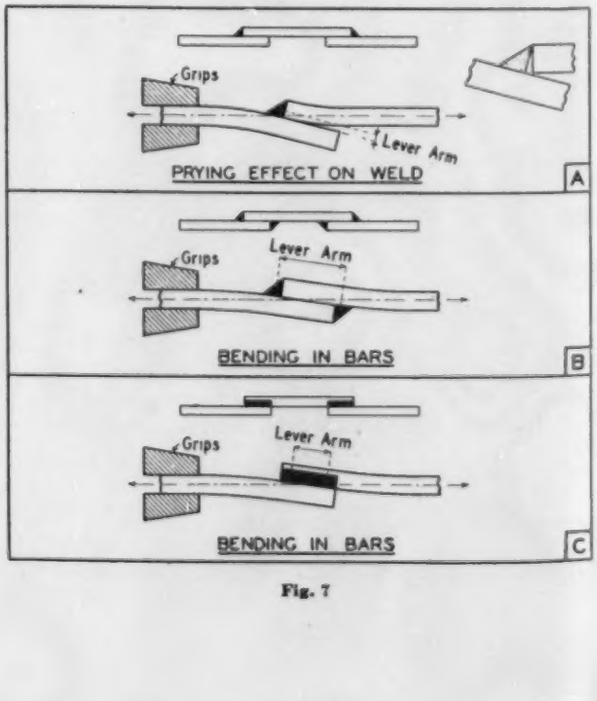


Fig. 7

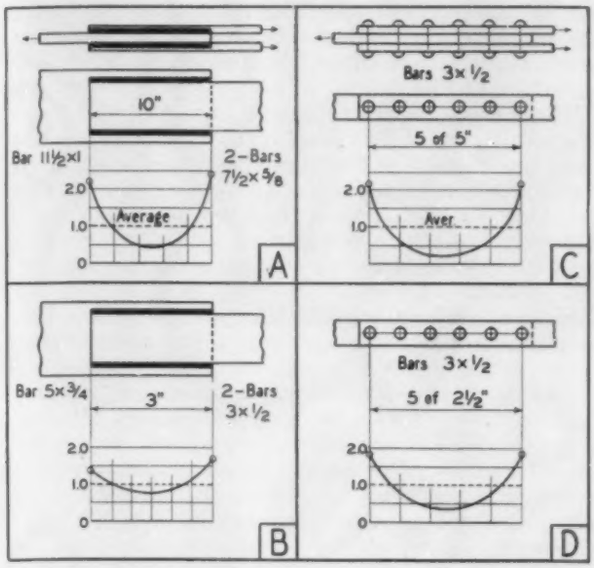


Fig. 8

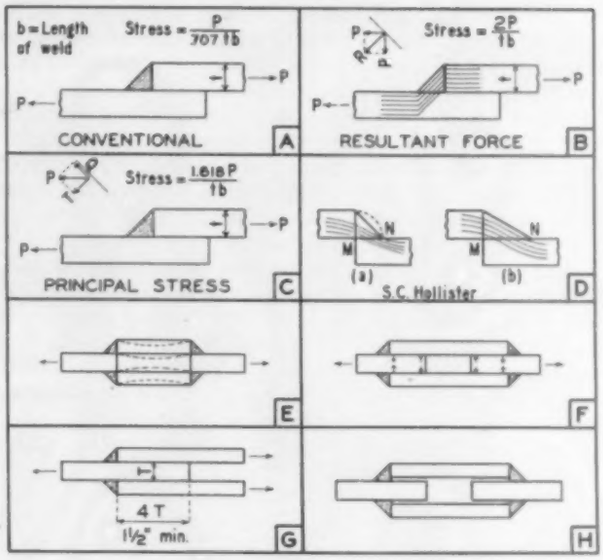


Fig. 9

As the throat is the critical section of both fillet and butt welds, it has become the practice to base the safe unit working stresses upon this section, as will be explained presently.

AMERICAN WELDING SOCIETY BUILDING CODE STRUCTURAL STEEL WELDING COMMITTEE REPORT

Every structural engineer should be acquainted with the "Code for Fusion Welding and Gas Cutting in Building Construction, Edition of 1930," formulated by the Committee on Building Codes of the AMERICAN WELDING SOCIETY. The code proper deals with definitions, materials, permissible unit stresses, design, erection and gas cutting while an appendix contains explanatory information and data.

For the purposes of this paper it will be useful to quote the following section of the Code:

"Section 4. Permissible Unit Stresses

1. Welded joints shall be proportioned so that the loads specified in the Building Code shall not cause stresses therein to exceed the following amounts in pounds per square inch:

Shear on section through throat of weld	11,300
Tension on section through throat of weld	13,000
Compression on section through throat of weld	15,000

Maximum fiber stresses due to bending shall not exceed the values prescribed above for tension and compression, respectively. Fillet welds placed transverse to the direction of stress shall be considered as under shear.

2. In designing welded joints adequate provision shall be made for bending stresses due to eccentricity, if any, in the disposition or section of the base metal parts."

Illustrations of the application of this section will be given in a later portion of this paper.

A companion document to the Code is the Report of the Structural Steel Welding Committee of the American Bureau of Welding, a Committee organized in 1926 for the purpose of "obtaining reliable information upon which to base safe unit working stresses in the designing of welded structures."

It is not possible to satisfactorily condense the findings of this investigation into the limited space which can be allotted to them in this discussion. A few of them are of special importance and immediately follow:

1. The unit working stresses, per square inch of weld throat, as recommended in the AMERICAN WELDING SOCIETY'S "Code for Fusion Welding and Gas Cutting in Building Construction, Edition of 1930," are safe and reasonable and are endorsed, provided the welder has been qualified and provision has been made for bending stresses that may be introduced due to eccentricity.

2. In commercial practice a welded joint may be expected to have a strength within 12% of a general average for that type of joint, provided the welder has attained the stated qualifications.

In the study of the test results it early became evident that eccentricity and its accompanying prying effect on the welds was an important factor in the strength of fillet welds. Figure 7 illustrates the effects of flexure on fillet-welded joints. For Sketch *A* it should be noted that the weld is subjected to a prying effect which is resisted by internal forces having a small lever arm. When welds are placed on the ends of both bars, as in Sketch *B*, the lever arm is materially increased and the bending is thereby thrown more largely into the bars The same is true of joints with side welds, typified by Sketch *C*.

Stress Distribution in Side Fillet Welds

The stress distribution in side fillet welds has received a very considerable amount of theoretical and experimental investigation. The late Professor J. Hammond Smith of the University of Pittsburgh seems to have been the first one to carry out a series of careful laboratory tests. These tests and a manuscript of a mathematical analysis by H. W. Troelsch (since published in A. S. C. E. Proceedings, November 1932) inspired a theoretical investigation by Messrs. Weiskopf and Male in which they confirmed the laboratory findings of Professor Smith. In a subsequent published discussion they developed several practical applications of the theory to structural joints. These applications are perhaps most valuable for comparing different types of connections rather than for the final design of a joint.

It had been surmised for some time that the stress

was not evenly distributed throughout the length of the weld. Professor Smith showed that the ends of a weld are more highly stressed than the central portion and Mr. Troelsch's analysis demonstrated the same fact.

Figures 8*A* and 8*B* show the theoretical distribution along the welds of two test specimens in the program of the Structural Steel Welding Committee. In the charts the vertical ordinates represent the intensity of stress on the basis of the average stress being equal to unity. Referring to Fig. 8*A* it will be noted that the ends have a stress from five to six times that of the central portion. (The variation in values at the ends is due to unequal areas in the component bars—causing higher concentration at the smaller bars.) The maximum stress is 2.44 times the average. It will also be seen that there is a large portion of the weld which has less than the average. Figure 8*B* is a similar chart for a specimen with only 3-in. welds and again we have the same general distribution but with less variation.

Any wide difference of distribution such as here shown attracts immediate attention and might possibly lead one to hesitate to use such welds. The publication of these various findngs was not accompanied by a sufficient amount of practical interpretation, with the result, no doubt, that many engineers misunderstood their significance.

It should be pointed out that all the results, theoretical and experimental, apply only within the elastic limit of the material. What undoubtedly happens is that yielding takes place within the steel bars and to a somewhat less extent within the weld itself. The result must be a flattening of the curve of stress distribution.

The Structural Steel Welding Committee conducted tests on 147 specimens of this type, with welds varying in size from $^1/_4$ in. to $^3/_4$ in. and in length from 2 in. to 12 in. The average ultimate shearing stress on the weld throat was 40,100 lb. per sq. in. with a minimum for any individual specimen of 29,800. Here is a large group of actual test results in which it is evident that an end stress of twice the average could not have existed at the time when the stress approached the ultimate or failure would have occurred at a much less average stress. With the working unit stresses recommended by the Committee the engineer may be confident of the design of ordinary connections to carry static loads.

An interesting paper by A. Hrennikoff, Esq., of the Dominion Bridge Co., on a similar investigation of the distribution of stress among rivets in a single line was published in the November 1932 Proceedings of the A. S. C. E. Figures 8*C* and 8*D* are charts of the theoretical distribution. The results are strikingly parallel to those for side welds. It has long been recognized that the end rivets were highly stressed so long as the material was not stressed beyond the elastic limit. Here again, the yielding action must produce a more uniform division of the stress among the individual rivets. Certainly no structural engineer hesitates to put several rivets in a line and calculate their number on the basis of equal strength.

The investigations do have a practical significance when one is designing a joint to carry impact or repeated stresses. Such loads might induce an initial fatigue failure at the extreme ends of the weld, followed by progressive failure with repetition of the loading conditions.

Stress Distribution in End Fillet Welds

Various theories or methods of calculation have been proposed for obtaining the stress developed in end fillet welds, that is, welds which are transverse to the direction of the pull. C. H. Jennings of the Westinghouse Electric and Manufacturing Company set forth three common methods in an article in the magazine, *Welding*, for February 1931.

These methods are shown in Figs. 9*A*, 9*B* and 9*C*. The unit stress on the throat of the weld is given without going into the derivation of the formulae. Mr. Jennings conducted a series of tests on actual specimens and compared the results with those given by the several methods, and from his investigation he concluded that the "Principal Stress Method," Fig. 9*C*, gave the most satisfactory results.

After all, the main reliance of the engineer must be tests to determine the actual loads which welds can carry. Many results of such tests are available and, with the proper factor of safety, the designer can rely upon the working unit stresses to give satisfactory structures. The complicated stress relations which undoubtedly exist within a weld make any attempts at exact analysis rather difficult, if not impractical. However, such attempts often point to certain features which are important.

Professor Hollister of Purdue University conducted some photo-elastic studies on celluloid models of fillet welds by means of polarized light. Figure 9*D* illustrates the results of his work. Sketch *A* is the usual 45-deg. fillet with the lines of stress flow. It will be observed that stress concentrations occur at points *M* and *N*. The upper corner of the weld does not receive much stress. Attempts to increase the strength of the weld by reinforcement to some such form as shown by the heavy dotted line are seen to be futile, as the stress will not flow out into the additional material.

Sketch *B* shows a 30-deg. fillet in which the weld surface is parallel to the general inclination of the lines of stress. It was pointed out that the critical section of end welds is the side of the weld which is in direct transverse shear and that "the 30-deg. fillet is not only more efficient, but provides an increased length of the transverse shear plane, thus adding to the safety of such a connection."

Although recognizing the desirable features of the 30-deg. fillets for end welds, there does not appear to be any necessity for changing from the practice of using 45-deg. fillets on all welds. The latter have shown excellent strength in tests on end welds and the additional strength afforded by 30-deg. fillets is not required.

"Thorough penetration at the root or throat of the fillet is, in the case of fillets in transverse shear or tension, of prime importance. Failure to achieve this penetration considerably increases the stress at this point." The lack of penetration is similar in its effect to a notch in the weld. The increase in stress is more a function of the form of the notch and is not so much a function of the reduced area of weld section which the notch causes.

While on the subject of stresses in end welds, it may be an appropriate point at which to make a few observations on connecting members by means of such welds. Figures 9*E*, 9*F* and 9*G* will serve to bring out some considerations which should be borne in mind. Professor Hollister's model was of the form in Fig. 9*E*, in which the central bars stop on the line of the weld. Such a condition would give rise to bending in the welds due to the tendency of the side or splice bars to deflect to the dotted positions. As he properly points out, the main plate should extend in between the side plates, so that the bending can be better resisted as indicated diagrammatically by the force couples in Fig. 9*F*. This is a very practical consideration when welding any structural members on their ends. The writer would suggest the requirements given in Fig. 9*G*. If the plates are not in

contact, as shown in Fig. 9*H*, then the conditions approximate those in Fig. 9*A*. Reasonable precautions should be taken to bring the surfaces into contact before welding.

Attachment of Structural Shapes

The attachment of members which are symmetrical about an axis parallel to the line of stress presents no serious problem. Figure 10 shows a few typical examples. The length of the welds should be equal on both edges and is determined by dividing one-half the total stress in the member by the unit working stress per inch of weld. On some occasions, for lack of space, it is necessary to place welding on the ends of a member as shown in Sketch *B*. The unit stress per inch for end welds is the same as for side welds. In such cases the value of the end weld strength is deducted from the total force, the remainder being divided equally between the side welds. The length of the end weld should not exceed the total length of the side welds.

When the member is composed of a single structural shape as in Sketches *A* to *D*, there is a certain eccentricity, *e*, the effect of which is to produce the condition shown in Sketch *E*. The member tends to distort until its center of gravity lies on the line of pull. For this reason, it is the general practice to reduce the unit stress in the member itself.

In a double member, Sketch *F*, each part resists the tendency of the other to bend so that we do not have the high bending stresses of the unsymmetrical member.

Attachment of Angles

The generally accepted method of calculating the length of welds required to attach angles is given in Fig. 11*A*. Assuming 2 Ls $3^1/_2 \times 2^1/_2 \times {}^5/_{16}$, the total allowable stress in one angle equals the area times the unit stress, i.e., equals $1.78 \times 18{,}000 = 32{,}000$ lb. The stress is divided between welds *A* and *B* inversely proportional to their distances from the center of gravity of the angles. The calculations are illustrated in the figure. It should be noted that the effective size of the weld at *A* cannot exceed the thickness of the angle, which, in this case, is ${}^5/_{16}$ in.

There is an eccentricity on the welds due to the center of gravity of the angles being a distance, *e*, from the back of the angles. It is desirable to keep this distance as small as possible and it has been the writer's practice to use unequal legs with the shorter one outstanding. With double-angle tension members it is essential to have separators to prevent their inward deflection and in struts to reduce the L/r ratio of the individual angles.

It has long been recognized in structural design that single angles in tension should not be used to their full cross-sectional value. A clause in the A. R. E. A., "General Specifications for Steel Railway Bridges," reads as follows:

"The effective area of single angles in tension shall be assumed as the net area of the connected leg plus 50 per cent of the area of the unconnected leg."

In welded construction there are no holes to be deducted for the net area of the connected leg. An approximation to the A. R. E. A. provision for the common sizes of unequal legged angles would be to take 80% of the gross area as effective.

Other factors in use vary from 60% to 75% of the gross area. Professor McKibben used 60% in an article in which he discussed single angle tension members. For angles with unequal legs the area in the longer leg equals approximately 60% of the total area of the angle, so that one rule might be set up by which only the attached leg of a single angle member in tension shall be considered effective. It has been the general practice to calculate the welds for single angles in the same way as for double angle members as outlined above.

Professor J. R. Griffith of Oregon State College takes sharp issue with this method of calculating welds on angles in a recently published article summarizing the results of elaborate stress measurements on single angle tension members tested under his supervision. The work was done by a graduate student, Mr. Howard Stanley, as a thesis for an advanced degree.

Figure 11*B* indicates the three types tested. The welds on *a* are calculated by the conventional method, on *b* they are equal in length and on *c* the conventional lengths are reversed. From the test results it appears that Specimen *b* gives lower stresses in the angle.

It seems to the writer that more corroborative evidence is needed before advocating the abandonment of a practice which is in general use in this country and in Europe. It is to be hoped that Professor Griffith

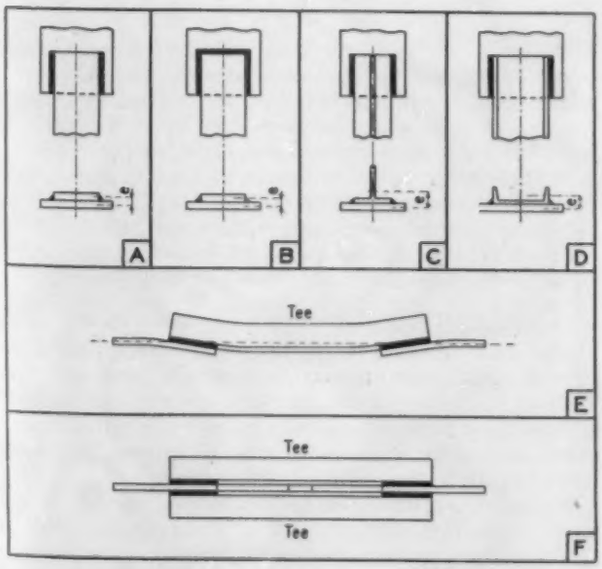


Fig. 10

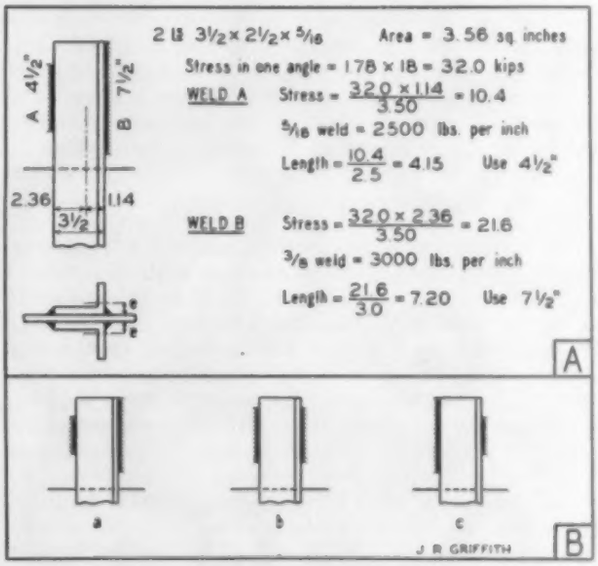

2 ls 3½ × 2½ × 5/16 Area = 3.56 sq. inches
Stress in one angle = 1.78 × 18 = 32.0 kips
WELD A Stress = 320 × 1.14 / 3.50 = 10.4
5/16 weld = 2500 lbs. per inch
Length = 10.4 / 2.5 = 4.15 Use 4½"
WELD B Stress = 320 × 2.36 / 3.50 = 21.6
3/8 weld = 3000 lbs. per inch
Length = 21.6 / 3.0 = 7.20 Use 7½"
A 4½"
B 7½"
2.36
1.14
3½
A
a
b
c
J. R. GRIFFITH
B

Fig. 11

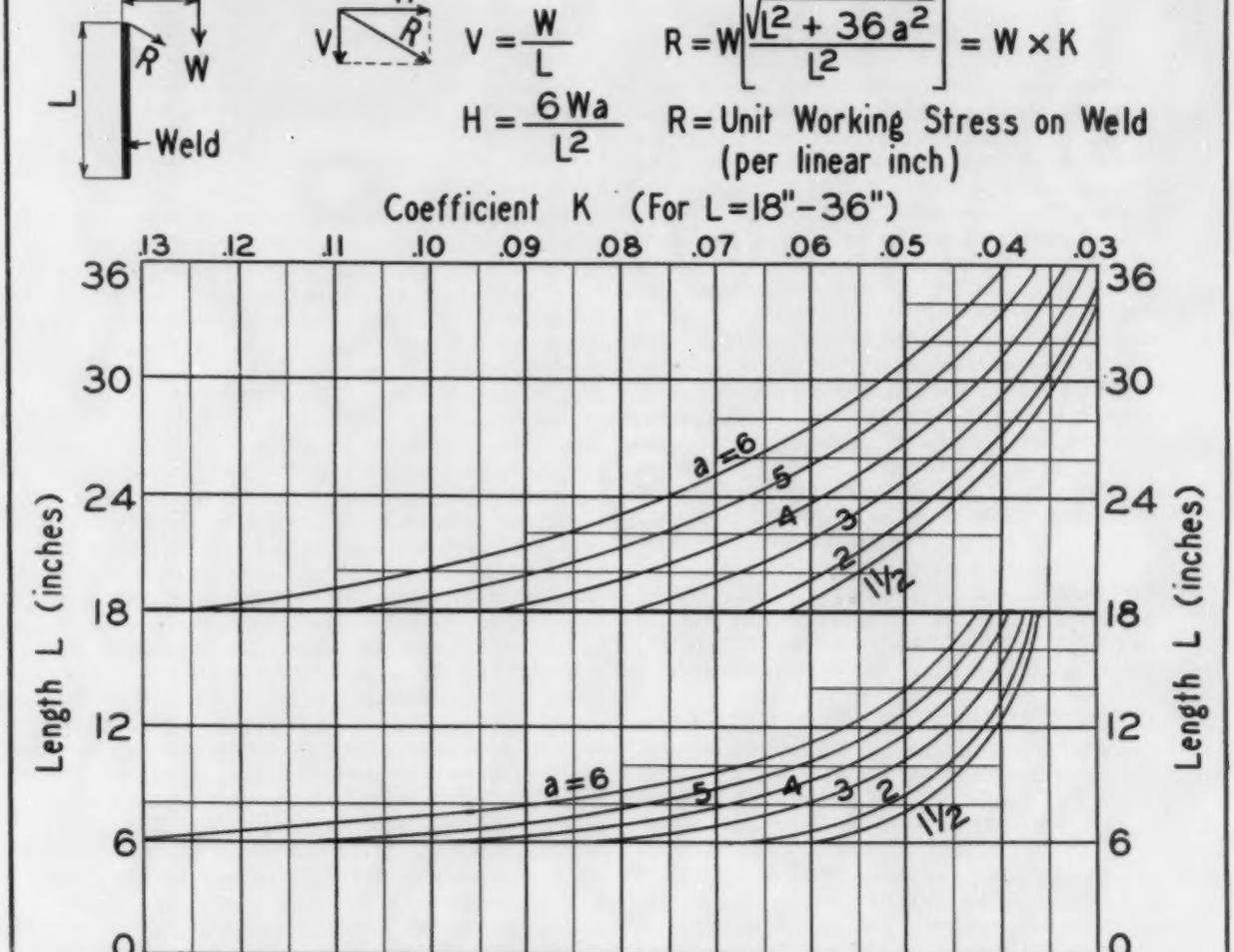

a
W
L
Weld
H
V
R
$V = \frac{W}{L}$
$H = \frac{6Wa}{L^2}$
$R = W\sqrt{\frac{L^2 + 36a^2}{L^2}} = W \times K$
R = Unit Working Stress on Weld (per linear inch)
Coefficient K (For L=18"-36")
.13 .12 .11 .10 .09 .08 .07 .06 .05 .04 .03
36 30 24 18 12 6 0
a=6 5 4 3 2 1½
Length L (inches)
1.0 .9 .8 .7 .6 .5 .4 .3 .2 .1 0
Coefficient K (For L = 6"-18")

Fig. 12

can carry out further research on this problem with a greater number and varying sizes of angles. In the meantime, it is suggested that engineers adhere to the conventional method which enjoys universal favor and has given satisfactory results in many structures.

Eccentric Connections

The accepted practice for designing eccentric connections is given in Fig. 12. Assume a weld of length, L, to be carrying a load, W, acting at a distance, a, from the line of the weld. The resultant stress on the weld is a combination of the vertical and horizontal components. The problem is simplified by treating only the linear dimension of the weld and then using a stress per linear inch instead of per square inch. The vertical component equals the load divided by the length of the weld, while the horizontal component equals the moment, Wa, divided by the section modulus of the weld, $L^2/6$. In the formula for the resultant stress, R, on the weld, it will be observed that the quantity within the brackets is constant for a given problem and has been given the value K.

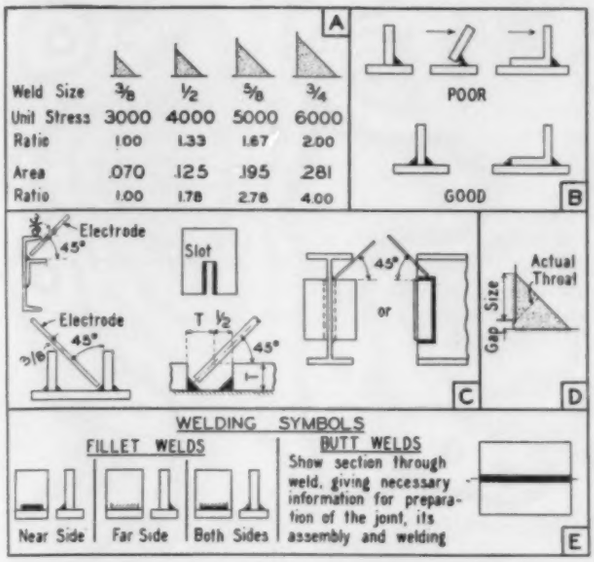
In order to facilitate the calculation of eccentric connections, the chart has been prepared, the use of which will be illustrated by the following example: Assume W = 45 kips and a = 4 inches. With a $^3/_8$-in. weld having a working unit stress, R, of three kips per inch, the value of K is found to be 3.0/45 = 0.067. Entering the upper half of the chart at K = 0.067 and following vertically down to the curved line for a = 4, we find that L should equal 22 in. A beam bracket with two such welds could sustain a load of 90 kips.

General Design Considerations

The chief difference in design between welded and riveted construction lies in the details of the connections. Welded joints are more rigid and require special study to avoid harmful effects which might follow from this condition. This discussion will, therefore, be devoted more largely to the joints of structures.

Early attempts at designing welded members consisted mainly in the straight substitution of welds for rivets. This was not satisfactory, practically or economically. A fresh approach to the subject has developed better methods.

Fig. 13



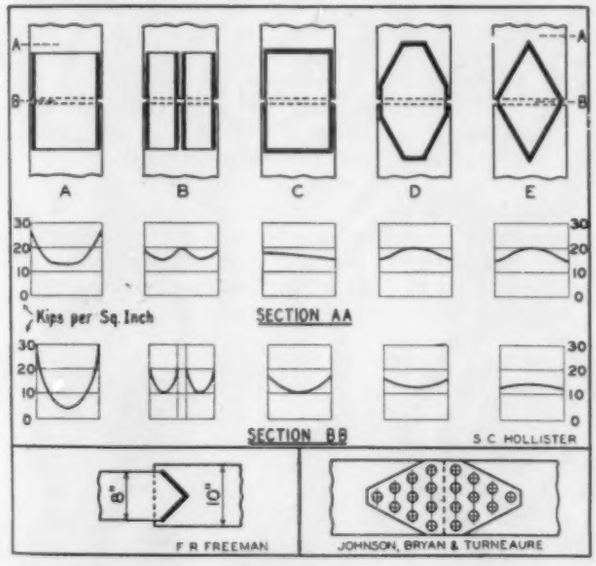


Fig. 14

A cardinal aim should be simplicity in the details. Study spent on the details before a job goes to the shop can effect worth-while savings in cost of welded construction. One fundamental difference from riveted construction must be constantly borne in mind. The shop assembly of riveted members is guided and facilitated by the punched holes. When welding is used, the aim is to avoid holes and the work is held together by clamps or jigs until the welds can be made. Much of the layout work is done directly on the steel without the usual templets. Dimensions should be given in such a way as to readily assist this part of the work.

Symmetry in the location of welds and welded connections is very desirable. The heat of welding is apt to cause a certain amount of warping, and symmetry will assist many times in maintaining the parts in their original or intended position or shape. The effect of a weld on one side of a member is to cause the member to take a curved shape, concave toward the weld. This condition is often avoided in the shop by securing the parts against bending during fabrication.

The easiest position for welding is downward. It is generally more comfortable for the welder and enables him to place the weld with greater uniformity. Vertical welding, in most cases, is quite easy to do, though not as fast as downward. Overhead welding is more difficult, both from the point of view of welding technique and from the uncomfortable position of the welder and furthermore, in the field, will usually require some form of staging or platform for the welder to stand upon. It is, therefore, very desirable to plan the design and the details of construction so that field connections may be welded in a down position, if at all feasible. Alterations and additions to existing structures may often require overhead welding, in which case it should be kept to a minimum.

The erection of structural steel requires some means of holding the members together until the field welds are placed. For members framing to columns, it is general practice to utilize a positive type such as a bolt or a drift pin. Minor connections have been made by clamps or wedges and in some instances without any temporary means. Examples of some of the methods used will be illustrated in a later part of this paper.

One principle of economical fabrication is to avoid, so far as possible, both welding and punching operations on the same piece of main material. This can often be accomplished by punching holes in the detail material which is welded to the large-sized members

Detail Design Considerations

The common sizes of electrodes for hand welding are $^5/_{32}$ and $^3/_{16}$ in. diam., while larger sizes are used on automatic machines. A $^3/_8$-in. fillet weld is most common in structural work for the reason that it is about the maximum size of weld which can be made in one pass with a $^3/_{16}$-in. electrode. Larger sizes of welds than $^3/_8$ or $^7/_{16}$ have to be made in two or more passes of the electrode in arc welding. In the gas-welding process any size can be made in one pass.

One point should be kept in mind when larger sizes of welds are used. The strength of welds is approximately in proportion to the throat dimension and consequently to the size, while the cross-sectional area of the weld varies as the square of the size. Figure 13*A* will illustrate certain facts in regard to size in which a $^3/_8$-in. weld is taken as a basis for comparison.

The area of the cross section of a weld is a measure of the amount of weld metal to be deposited. To go from a $^3/_8$-in. to a $^1/_2$-in. weld increases the strength by the factor 1.33, while the weld metal required is nearly doubled. Weld metal is deposited at a uniform rate, hence the total time of welding increases more rapidly with larger welds. The effect on the cost is evident. The cost of depositing a pound of weld metal varies very little for the usual sizes of welds. Therefore, it is obvious that the ratio of unit stress to area should be kept large.

The minimum size of fillet welds for strength purposes should be limited to $^1/_4$ in. and the length of no weld should be less than four times its size. For example, the minimum length of a $^3/_8$-in. weld should be $4 \times {}^3/_8 = 1^1/_2$ in. With butt welds the least permissible length is commonly taken as four times the throat dimension.

The distribution of welding is often of prime importance when it relates to the ability of a joint to withstand the handling to which it is subjected in shipping and erection. A connection which is entirely adequate to perform its final function in the structure may be wholly unsatisfactory for handling.

Figure 13*B* will clearly bring out the difference between poor and good construction. When a member is

welded on one side only, the connection has very little resistance to a force acting in the direction of the arrows. By placing welds on the opposite edge, an almost surprising amount of stiffness is gained. A case came to the writer's attention in which an angle was satisfactorily stiffened for shipment by placing welds 2 in. long about 5 ft. 0 in. ctrs. on the toe of the angle.

No design will be completely satisfactory unless attention is given to arranging the details so as to provide sufficient clearances for performing the welding. Just as there are considerations in riveted work which affect gages and spacing of rivets, so there are certain points which should govern in welded construction. First of all, the welder must be able to see his work. Secondly, it must be possible to hold the electrode or welding rod in the proper position to secure penetration and sound quality in the welds.

A few typical cases shown in Fig. 13*C* will illustrate the requirements for arc welding. The requirements for gas welding are of a similar character, bearing in mind that space must also be provided for manipulating the welding torch.

The ideal position of the electrode when welding material of approximately the same thickness is at 45 deg. to the legs of the fillet. It is a simple matter to make a quarter scale layout of joints when necessary to determine the clearance. A practical rule for the width of slots is to make the opening equal to the thickness of the material plus one-half inch.

One practical remark should be made about welding across a small gap. When such a case arises, the size of the weld must be increased by the amount of the gap in order to provide the designed size of throat. Figure 13*D* shows this requirement.

Those who were connected with the design of welded structures in the early stages of the development felt the immediate need of some method of showing the welds on the drawings. Structural work differs so widely from shipbuilding practice in its drawing room and shop methods that the simpler symbols for welds on the near and far sides shown in Fig. 13*E* have found quite general use in place of the more elaborate symbols used in ship work. Recently the symbol for welds on both sides has been proposed and has considerable merit, as it is a combination of the separate symbols. The form of joint for butt welds is so varied and their use so limited in structural work that it has been found best to indicate the weld by a heavy line and give the details of the joint in a separate section.

Butt-Strap Splices

An excellent paper by Professors Hollister and Gelman of Purdue University entitled, "Distribution of Stresses in Welded Butt Strap Joints," was presented at the 1932 Fall Convention of the AMERICAN WELDING SOCIETY. Figure 14 gives the general results of their investigation on five types of splices with double straps. The increment of load for which the stresses were determined by means of Huggenberger extensometers was 50,000 lb. On the basis of a uniform distribution, the stress in the plates is as follows:

Main plate (all types) = 16,670 lb. per sq. in.

Butt straps (Types *A*, *C*, *D* and *E*) = 14,290 lb. per sq. in.

Butt strap (Type *B*) = 16,670 lb. per sq. in.

It is readily apparent that where some of the welding is transverse to the line of stress, as in Types *C*, *D* and *E*, there is a more uniform distribution of stress on Sections *AA* and *BB* with decreased intensity. Type *E* gave the best all-around results. From the standpoint of ease in preparing the material, the butt-straps of Type *C* are the simplest. For wide members, the Type *B* splice is a good one.

The authors state that the data refer only to stresses produced in the specimens by direct tension loads and that the relative suitability of different types of splices may be affected by a change in the method of loading.

Structures or machine parts subjected to repeated loads require special consideration where fatigue stresses may occur before the yield point is reached. Here "the knowledge of the distribution of stresses in various types of welded connections will help to select the proper detail for each particular case."

In an investigation reported by F. R. Freeman, Esq., before the Institution of Civil Engineers in England, one type of specimen was that shown in the lower left-hand corner of Fig. 14. He stated that, "This type of joint would appear to be very suitable for connections where it is practicable, as the stress tends to be very evenly distributed over the weld and plate with a short length of weld."

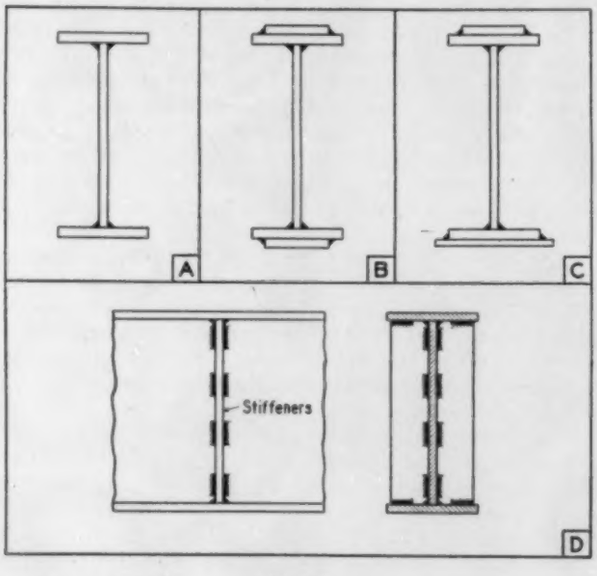


Fig. 15

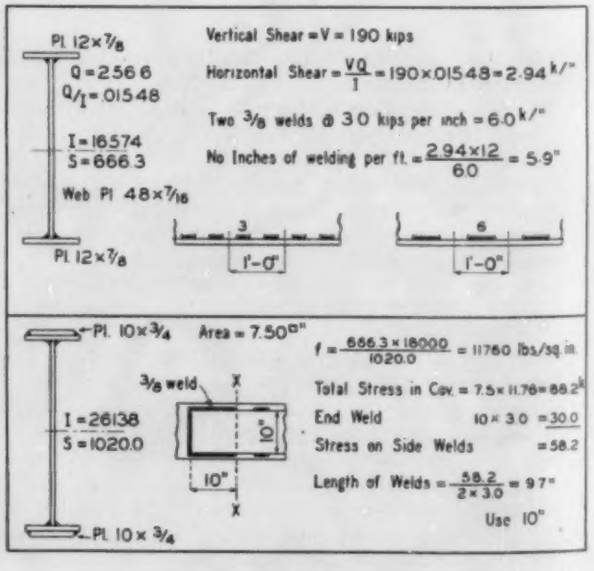


Fig. 16

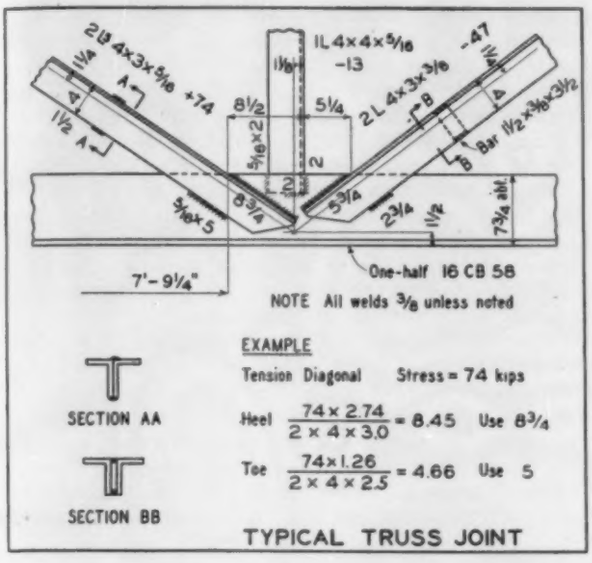


Fig. 17

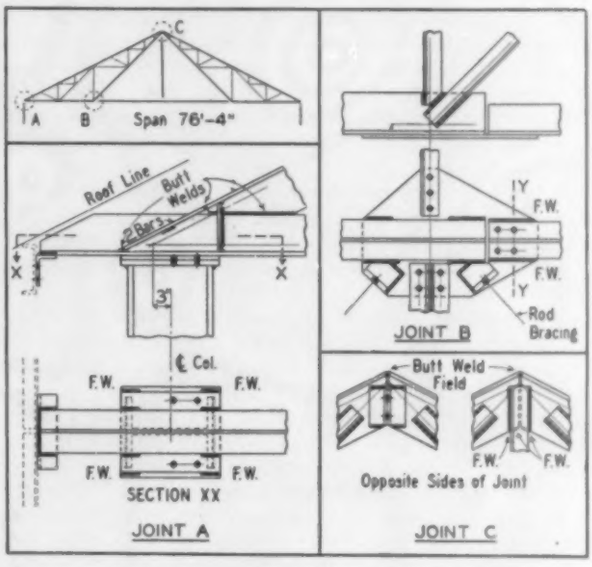


Fig. 18

It is interesting to note that in "Modern Framed Structures," by Johnson, Bryan and Turneaure, a riveted splice with tapered splice plates (see Fig. 14) is shown to give a better distribution of stress than a square joint.

Plate Girders

The simplest form of welded plate girder is shown in Fig. 15*A*. It consists of a web plate, as in the usual riveted plate girder, but the flanges are composed of plates which are welded directly to the web. When it is necessary to increase the flange area it may be done by adding cover plates as shown in Figs. 15*B* and 15*C*. It is quite general practice, especially when strengthening existing beams or girders, to make the bottom flange cover wider than the flange plate and the top cover narrower than the flange plate in order that the welding on the covers may be performed in a downward position. Stiffeners can be bars attached to the web with chain intermittent welds and to the flanges by welds on one or both sides of the bars as illustrated in Fig. 15*D*.

The calculations for the welding on a plate girder are given in Fig. 16. The properties of the cross section have been found by the usual methods and are as follows:

Moment of inertia, $I = 16{,}574$

Section modulus, $S = 666.3$

Statical moment of flange, $Q = 256.6$

Assume the vertical shear, V, to be equal to 190 kips. From the well-known formula the horizontal shear between the web and the flange plates $VQ/I = 2.94$ kips per in. or 35.3 kips per ft. of girder. Two $^3/_8$-in. welds at 3.0 kips are good for 6.0 kips per in. Hence, in one-foot length of girder there are required 35.3/6 = 5.9 in. of welding, or practically 6 in. Two alternative arrangements of the welding are shown. The 3-in. welds at 6-in. centers give a more uniform distribution of shear on the web.

At the ends of any girder it is desirable to make the welding continuous for a reasonable distance even though the design calculations do not require it. Where the girder will be exposed to the weather it is essential to seal the joints with a light fillet weld which can be put on between the required strength welds. This is open to the objection that the sealing welds will tend to take as much shear per inch as the larger welds and so be over-stressed. In some cases it may be advisable to run the $^3/_8$-in. welds continuous or perhaps reduce the size to $^5/_{16}$ in. for the continuous welds.

As in riveted construction, covers should extend past the point at which they are required in order to develop their stress. Assume 10 x $^3/_4$ covers added to the plate girder in the upper half of Fig. 16. The fiber stress in the original girder flange will be assumed as 18,000 lb. per sq. in. when the covers are needed. The necessary calculations are shown in the sketch with a result which calls for the covers to extend 10 in. beyond the theoretical point designated by the Line XX.

The design of stiffeners and their spacing can follow the usual structural practices. The welding on bearing stiffeners should be sufficient to transmit the load from the stiffeners to the web plate.

Trusses

The design of welded trusses has brought about the use of split-beam chord sections, the beam webs acting as the gussets to which the truss members are welded. A typical joint is shown in Fig. 17. The calculations for the lengths of the welds on the tension diagonals are given and follow the principles set forth in Fig. 11. The center of gravity lines of the angles meet at a common point in order to avoid eccentricity. The size of the beam selected for the chord section depends upon two things, first it must have the required area and second the stem formed by splitting the beam on the center line of the web must be wide enough to accommodate the welding required on the diagonal members.

The thickness of the stem should not be less than $^1/_{24}$th of the distance from the beam fillet to the edge of the stem. When double members are connected with welds on opposite sides it is necessary to examine the stem for shear. The following table gives suggested minimum thicknesses of stem for varying sizes of welds:

Weld Size, In.	Min. Thickness, In.
$^1/_4$	0.17
$^5/_{16}$	0.21
$^3/_8$	0.25
$^1/_2$	0.33
$^5/_8$	0.42

The 16 CB 58 has a web thickness of 0.375 in. and the

stem is, therefore, amply strong against the shear from $^{3}/_{8}$-in. welds on opposite sides which require a minimum thickness of 0.25 in. The distance from the fillet to the edge equals 7.75 − 1.27 = 6.48 in. and $^{1}/_{24}$th of this distance = 0.27 in., the minimum thickness which should be used. Again the 16 CB 58 satisfies the limiting conditions.

Section *AA* illustrates one method of tying double-angle tension members together. Compression members should have fillers (equivalent to washers in riveted work) as shown in Section *BB*, and fillers can also be used in tension members.

Figure 18 shows the important joints of an ordinary Fink-type truss. Because of its size the truss was shipped in sections with field splices at Joints *B* and *C* and at the bottom of the vertical center member. The truss was assembled on the ground and the splices welded before erection. Split beams were used for the top and bottom chords and the web members were angles. It will be noted that the bottom chord between Joints *A* and *B* has no holes punched in it so that the gross area can be used in tension. The holes for the erection bolts connecting the truss to the top of the column were placed in a plate welded to the under side of the bottom chord. A brief examination of the details of Joint *A* will show how the stresses were transferred.

Joint *B* shows the bracing gusset plate which was shop welded to the bottom chord of the half-truss. The center section of the chord had no shop-welding operations so it was decided to punch the erection assembly holes directly in the member. The first holes on Section *YY* were located far enough back from the ends of the field welds so that the stress on the net section would be reduced below the working value by the amount of stress removed from the member by the welds before the section was reached.

The peak of the truss is shown in Joint *C*. The assembling of the truss sections was facilitated at this point by shop welding a short angle to each half of the truss with holes to match. The two field bolts drew the parts quickly into position and alignment. Another point of interest is the splicing of the chord flanges by a field butt weld. The erector reported that this splice added a great deal of lateral stiffness to the joint during handling and erection of the truss. The stems were spliced by means of the vertical angle member, the attached leg serving practically as a splice plate. The toes of the assembly angles were also welded together. In order to guard against excessive stress in the edge of the stem due to the truss deflection, the edges were welded to the vertical angle as shown.

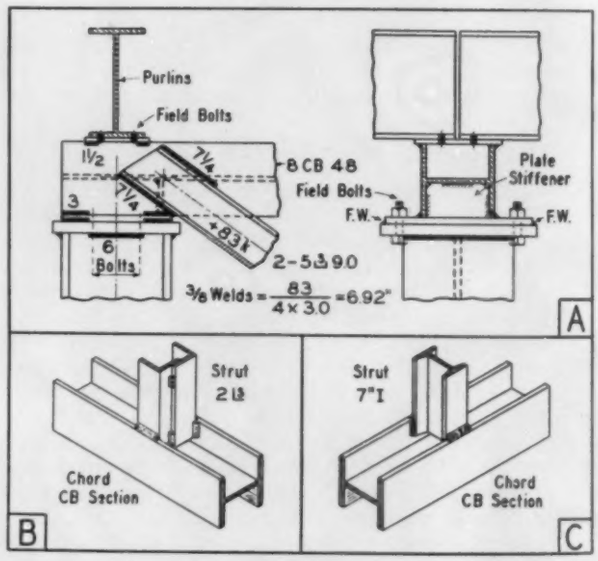
Figure 19*A* is a joint from a heavy truss of the Pratt type in which the chords were *CB* sections. The diagonals were channels and the compression verticals were either angles or 7-in. I-beams as shown in Figs. 19*B* and *C*. The welds on the channels were calculated according to the method explained with Fig. 10. A small plate stiffener was welded between the *CB* flanges to add support to the flanges and assist in transferring the reaction to the column. Holes for erection were kept in the detail pieces welded to the main material. This avoided extra handling of the heavy chord sections to the punches in the shop and illustrates the point previously emphasized that the combination of welding and punching on the same main member is not desirable.

Column Bases

The simpler forms of column bases may be divided into two classes—one in which the base plate is shop-welded to the column and the other in which the base plate is shipped separately and set in place on the masonry before the column is erected. Examples of both types are shown in Fig. 20. *A* and *B* show two typical arrangements of the shop welding. The first is the simpler from a fabrication standpoint but the latter is a more effective distribution of the welds and is recommended as preferable. A ready rule for the amount of welding around each flange toe of Base *B* is to make the total length equal to one-half the width of the flange. This makes the amount of welding the same on both bases. Web welds, as shown in *A*, are not necessary on small columns but are desirable on wide columns such as occur in industrial mill buildings.

Bases *C* and *D* illustrate two methods of shop-welding base angles to a column when the base plate is shipped loose and set in advance. The choice of details depends upon the location of the anchor bolts. In *D* the angles are set out from the web at least far enough to clear the column fillets and are welded to the column flanges. In both types the angles should be set slightly (perhaps $^{1}/_{32}$ in.) back from the finished end of the column

Fig. 19



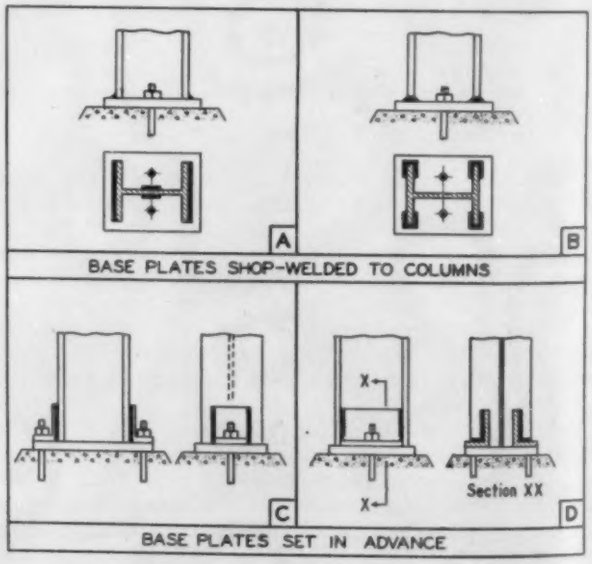


Fig. 20

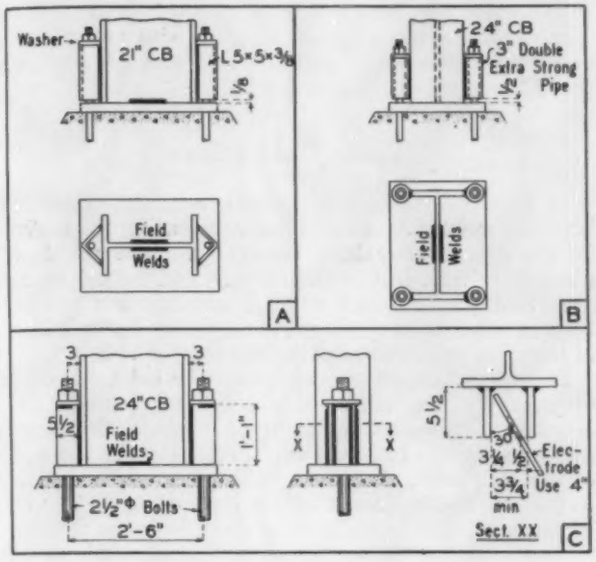


Fig. 21

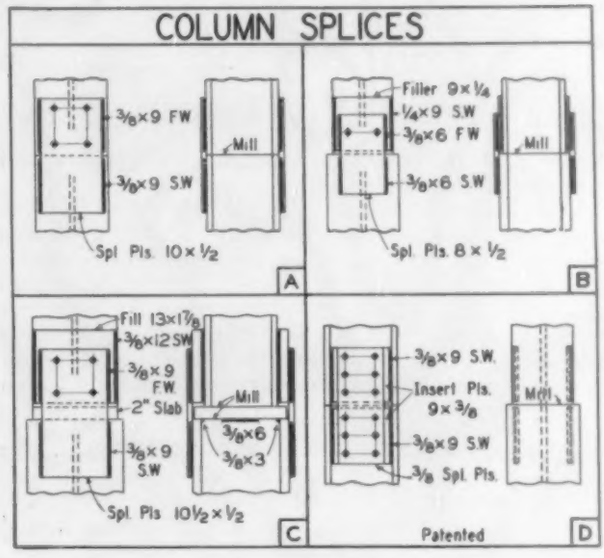


Fig. 22

to insure against the possibility of their bearing on the base plate and so throwing stress on the welds.

A more important class of column bases, from the point of view of design, is that in which provision must be made for uplift. These are the bases which require the familiar "boot" construction often found on such structures as mill buildings and crane runways. Figure 21 shows several forms of welded construction.

In Base *A* 5 x 5 x $^3/_8$ angles were welded to the column shaft and the anchor bolts located so as to have about $^1/_8$-in. clearance from the inside faces of the angles. The anchorages on Base *B* were made from pieces of 3-in. Double Extra Strong Pipe gas welded to the edges of the column flanges. It will be noted that in both types the angles and the pipes are set back from the finished end of the column to avoid any possibility of the direct column load being carried on the boots.

Base *C* is still another type in which bar stiffeners in pairs are welded to the column with a small bearing plate welded to their upper ends upon which the nut rests. As in Bases *A* and *B*, the usual practice was followed of setting the base plate in advance of the steel erection.

Without going into the design of a structure let it be assumed that the calculated uplift on one of the anchor bolts in Base *C* is 90 kips. The essential features surrounding the design of this anchorage will be illustrated in the following calculation:

At 18,000 + 33$^1/_3$% = 24,000 lb. per sq. in.

Required area of bolt = 90/24 = 3.75 sq. in.

Net area of 2$^1/_2$ bolt = 3.72 sq. in.

Since the long diameter of a hexagonal nut is 4 $^1/_2$ in. the center of the bolt must be out from the column face at least 2$^1/_4$ in. + $^1/_2$ in. for clearance. In order to make the center to center distance of anchor bolts an even figure let the bolts be placed 3 in. from the face of column.

The length of the stiffeners is determined by the amount of welding required to transfer the load of 90 kips from the column to the bolt. Since there are four lines of vertical welds, each weld will carry 90/4 = 22.5 kips. The chart in Fig. 12 will give the answer when the coefficient is calculated,

$$3.0 = 22.5 \times K$$

$$K = 0.133$$

For this value of K and an eccentricity of $a = 3$ in., it is found that 13 in. of welding is required.

The top bearing plate must be at least 5$^1/_2$ in. wide in order to provide a seat for the nut and the stiffeners will be made of this width. With this width fixed, the minimum clear distance between stiffeners can be determined as shown in the sketch. Although 3$^3/_4$ in. is indicated, it would be better to use 4 in. as it provides slightly more clearance for the electrode and is an easier measurement for the shop.

In all bases in which uplift is a factor especial attention should be given to the horizontal shear on the column and the shaft welded to the base plate (either in the shop or field) with sufficient welding to transmit this shear.

Column Splices

In tier buildings where the column splices serve more particularly to hold the abutting sections in line and not to transmit stresses, it will be sufficient to substitute welding for riveting directly. For erection purposes it is necessary to provide bolt holes so that the parts can be held in place until the field welding is done. Figure 22 shows the splices which were used in the nineteen-story office building of the Dallas Light & Power Company in Dallas, Texas, as described by Prof. McKibben in *The Welding Engineer*.

These are excellent examples of what might be considered good practice. If any suggestion of change were to be made it might have been possible to shorten the filler plates 3 in. and place 3 in. of horizontal welds on each side of the ends of the fillers. The balance between the welds on the fillers and the upper half of the splice plates is worthy of comment. In splice *B* 6 in. of $^3/_8$-in. welds at 3000 lb. per in. are worth 18,000 lb. and 9 in. of $^1/_4$-in. welds at 2000 lb. are also worth 18,000 lb. In Splice *C*, 9 in. of $^3/_8$-in. welds at 3000 lb. per in. are worth 27,000 lb. This force is acting 1$^7/_8$ in. from the line of welds on the edges of the filler plate. By referring to the chart, Fig. 12, and calculating the

value of K we can determine the required length of welding.

$$3.0 = 27 \times K \qquad K = 3/27 = 0.111$$

For $a = 1^7/_8$, $K = 0.111$. Required $L = 12$ in.

Figure 22*D* shows a splice which avoids fillers and punching of column flanges.

Another class of column splices is to be found in crane columns of mill buildings in which bending moments enter into the problem. A typical example is shown in Fig. 23. The building in which this occurred was designed on the basis of 16,000 lb. per sq. in. in tension on the steel, in accordance with the governing building ordinances. On this same basis the splice was calculated for the allowable bending moment on the upper 14 CB 95 section.

14 CB 95 $S = 150.5$ $M = 16 \times 150.5 = 2410$-in. kips

Flange stress = 2410/14.19 = 170 kips

Outside Splice Plate.—Required area = 170/16 = 10.6 sq. in. Plate $10^1/_2$ x 1 = 10.5 sq. in. In order to avoid an excessive length of splice it was decided to use $^5/_8$ welds which have a value of 5000 lb. per lin. in.

Required length of welds = 170/(2 × 5) = 17.0 in.

Inside Splice Plate.—The same size, $10^1/_2$ x 1, was used and slotted for the web of the lower section, 24 CB 100. The welding on the upper half was made the same as on the outside plate On the lower half the shear on sections *XX* and *YY* of the 24 CB web was investigated on the basis of 10,000 lb. per sq. in. To resist the flange stress of 170 kips required a length = 170/(2 × 10 × 0.45) = 18.9 in. As this was practically the length of the outside splice plate below the splice line it was made the same. Four $^3/_8$-in. welds 17 in. long at 3000 lb. per in. have a total value of 4 × 3 × 17 = 204 kips which is in excess of the requirements of 170 kips.

Figure 24 is a similar splice in which the inside flange of the upper section is spliced by means of two plates, one on either side of the web. The edges were beveled so as to clear most of the fillet and an over-sized weld used to make up for the gap. A horizontal weld was placed along the joint line to carry the horizontal shear. The outside flange splice was made with two plates in order to shorten the over-all length of the splice by using four lines of welds. (See Fig. 14.)

Beam Connections

It is probably safe to say that beam connections have been the most difficult problem confronting the engineer in the design of welded structures. The problem is not yet settled, but enough study and actual practice are available to point to safe construction methods. Cost of fabrication requires consideration and will be one of the principal factors in the final solution.

In approaching the subject it may be helpful to review very briefly the action of a beam when under load. Assume a beam to be simply supported (free-ended) as shown by the full lines in Fig. 25. Under the action of vertical loads, the beam will deflect an amount, Δ, to the position indicated by the dotted lines. The ends of the beam will be inclined by the amount of the angle, θ, and the top flange displaced a distance, b, with relation to the bottom flange. The table gives the formulae for these quantities for three of the most common loadings. Two sets of formulae are given: one when the maximum fibre stress in bending is 18,000 lb. per sq. in., the other when the center deflection equals 1/360 of the span.

The function of a beam connection is to transfer the end reaction or shear to the supporting member, be it a column, plate girder or another member. Figure 26*A* illustrates one type of connection which has undoubtedly suggested itself to many on first thought. The end of the beam is butted against the column and the web welded directly for the shear. Apart from the question of the adequacy of the connection there is a practical objection to this method. The welding might span a gap of perhaps $^1/_8$ in. as in Fig. 26*B*, but even with this tolerance it would be extremely difficult to keep within that limit unless the beams were finished to length—an expensive operation. Even then the erector would have to contend with over-run on columns, inaccuracies of location due to play in anchor bolt holes and other causes readily evident to those familiar with field work. This difficulty can be overcome by the use of bars on

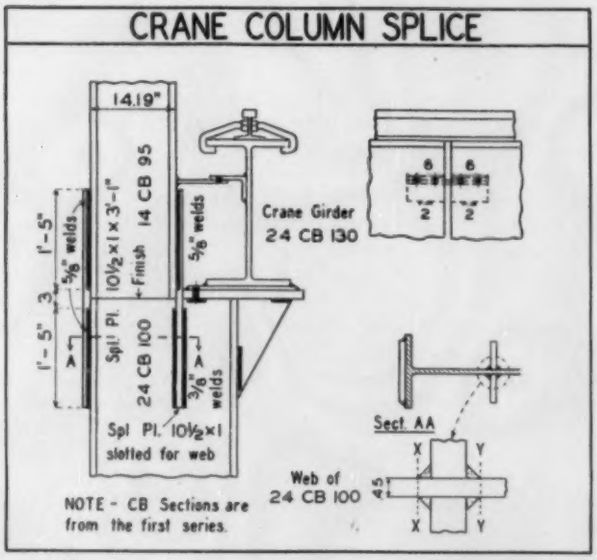


Fig. 23

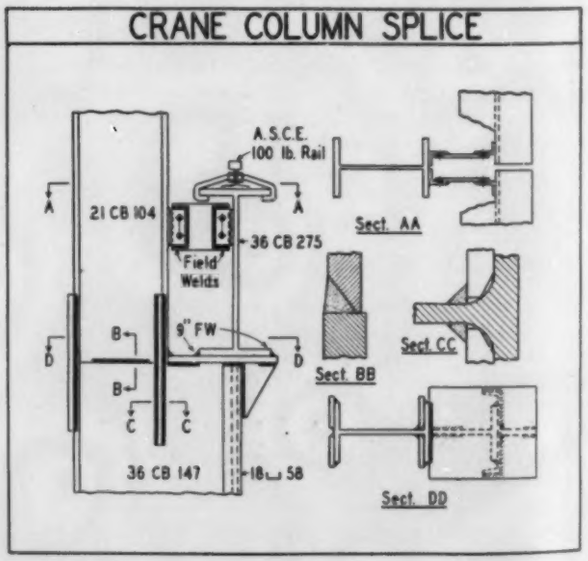
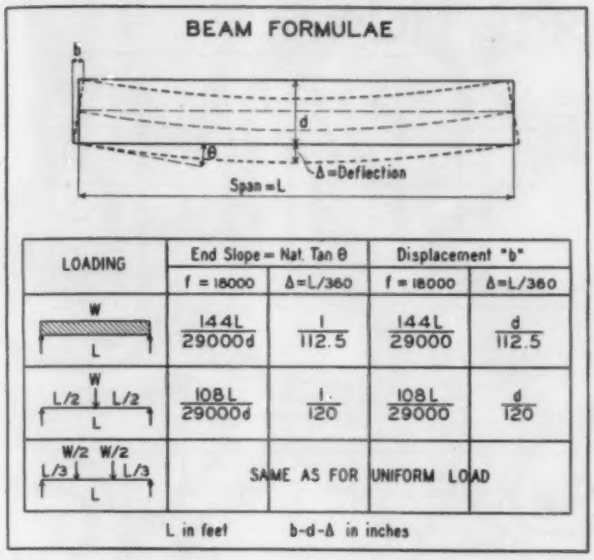


Fig. 24

Fig. 25



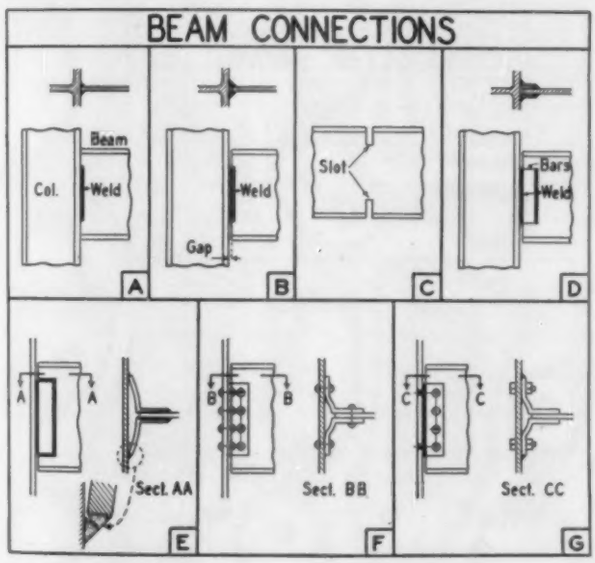


Fig. 26

either side of the web as in Fig. 26*D*. However, some such device as an erection seat would have to be provided to support the beam until welded.

The connection should be analyzed for strength. As an example, take an 18 CB 47 beam on a 30-ft. span for which the total uniform load is 34,200 lb. at a bending fibre stress of 18,000 lb. per sq. in. The end reaction is then 17,100 lb. which requires 17,100/3000 = 5.7 in. of $^5/_8$-fillet weld, or practically 3 in. on each side of the web. Assume that we have the condition in Fig. 26*B* and the weld increased in size in accordance with Fig. 13*D*. From the table of Fig. 25 the end slope of the beam is found to be 0.00828. If the rotation takes place about the center lines of the welds, the displacement at the top of the welds will be equal to 0.00828 × 1.5 = 0.0124 in. Recalling that structural steel at the yield-point stress increases in length about 0.001 in. per in., it is apparent how seriously over-stressed the weld and the adjacent beam web would be. The welds tend to fix the end of the beam by their resistance to the deformation, but they are wholly inadequate to do so. The condition at the connection is much the same as would exist in Fig. 26*C*, were slots to be cut to a depth such as to leave a width of web equal to the length of the welds.

Laboratory tests of this type of connection have been carried out by Professors Uhler and Jensen of Lehigh University and Messrs. Jennings and Jakkula of the Westinghouse Electric and Mfg. Company. Professor Jensen reported factors of safety on the working load ranging from 0.3 to 3.2 and both investigations revealed the dangerous character of this type of connection when used alone.

It has been suggested that the welds be made heavy and so force the deformation to take place in the web. This seems, at best, a rather doubtful expedient. The reasonable and safe procedure is to avoid this type of web connection unless the beam flanges are sufficiently held as to fix the end of the beam and consequently remove the moment from the vertical web welds.

A type of connection which has been used is shown in Fig. 26*E*. It is an attempt to utilize the flexibility of the outstanding legs of the connection angles to provide for the turning motion of the end of the beam. (All structural engineers recognize that such an action takes place in the ordinary riveted connection of Fig. 26*F*.) This welded connection should be used with caution, especially when the angles are 6 in. or more in length. The enlarged view of the weld will illustrate the fact that the deformation of the angle tends to produce tension at the root of the weld, a condition which should be avoided, especially where variable stresses are frequent in cycle.

This same type of connection can be utilized with much greater safety if erection bolts are provided near the top of the angles and left permanently in the structure. In the 15-story building of the Edison Electric Illuminating Company in Boston a typical framing connection is that shown in Fig. 26*G*. Although shop-riveted, all the field connections were welded. The erection bolts were permanent and serve to take the bending stresses, and the welds carry only the vertical shearing loads. A similar all-welded connection was used on the 19-story building for the Dallas Power and Light Company in Dallas, Texas.

The design calculations for the welding on such a connection are explained in Fig. 27. The problem is to find the lengths for the shop and field welds. The shop weld for one angle is shown in Fig. 27*C*. The first step

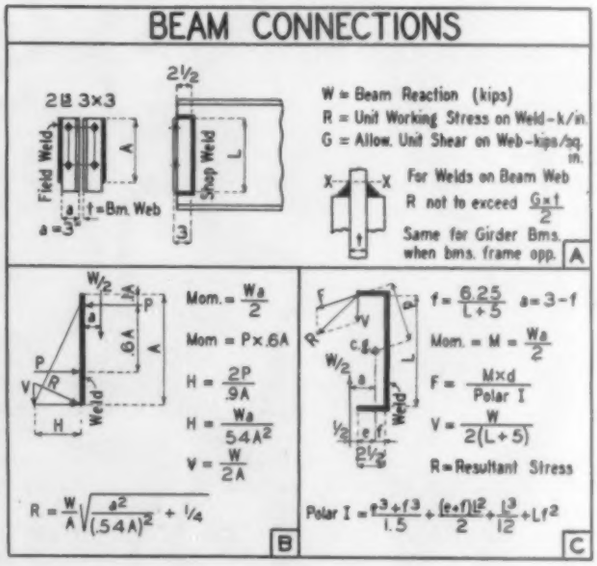


Fig. 27

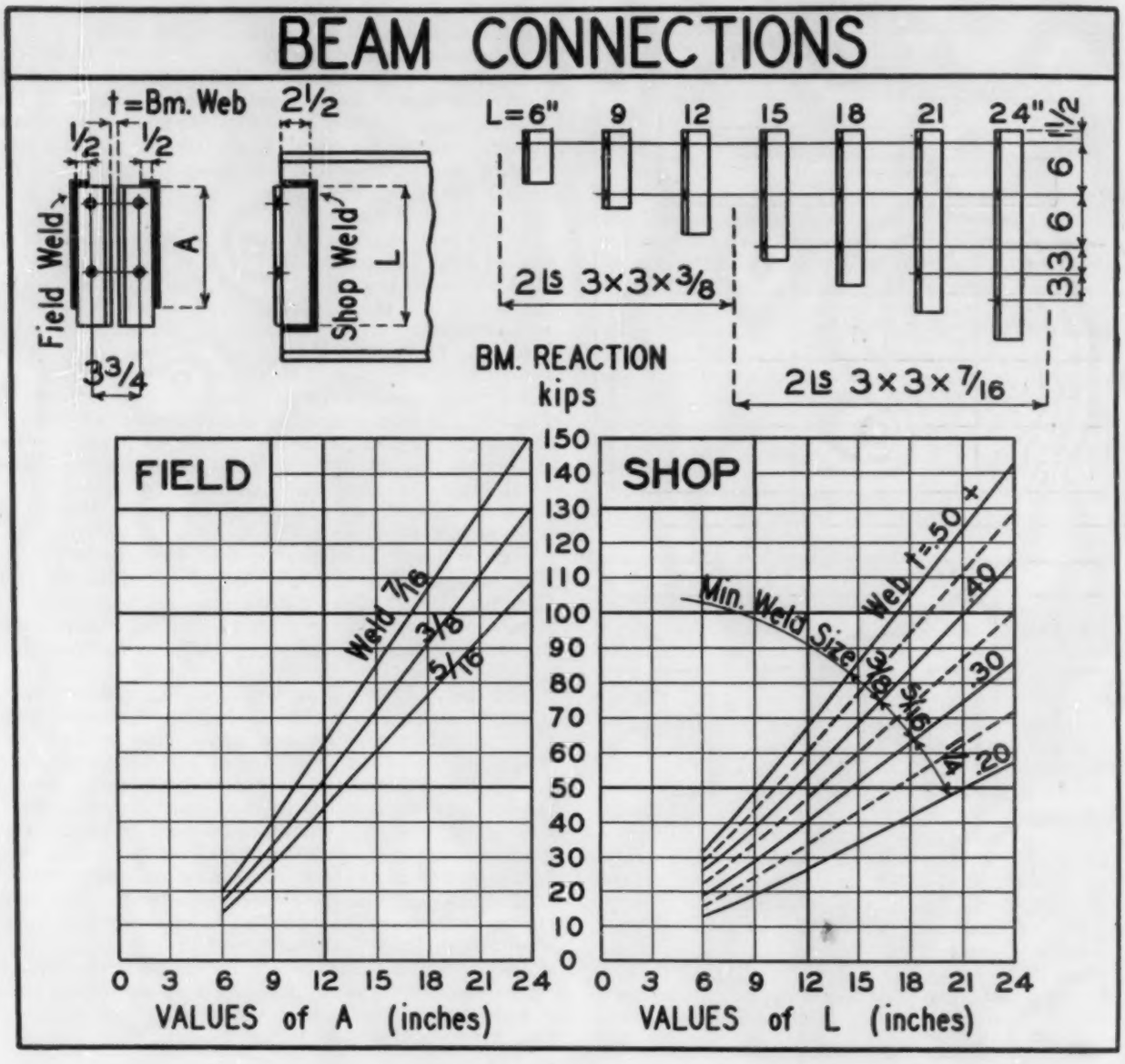


Fig. 28

is to locate the center of gravity of the weld and then compute its polar moment of inertia. (In all calculations of this kind the writer has found it simpler to treat the weld as a line and then use a working stress for the weld in terms of lb. per in. This avoids the introduction of the throat dimension and simplifies changes in size of the weld.) The essential steps are indicated for obtaining the components F and V of the resultant stress, R, on the welds.

It is evident that the web thickness should enter the problem, just as in riveted construction. In the upper half of Fig. 27, imagine the Section XX to be taken through the web just above the horizontal welds on the top ends of the connection angles. If the working strength of the two welds is limited by the shearing stress on the web, then $R = G \times t/2$ for each weld. Using a shearing stress, G = 12,000 lb. per sq. in. the following values of R are given for various thicknesses of beam web, assuming $^3/_8$-in. welds as the maximum size.

It is evident that $^1/_4$-in. welds at 2000 lb. per in. can be used up to t = 0.30 in., $^5/_{16}$-in. welds up to t = 0.40 in. and $^3/_8$-in. welds for all values of t above 0.40 in.

t, Inches	R, Lb./In.
0.50 & over	3000
0.45	2700
0.40	2400
0.35	2100
0.30	1800
0.25	1500
0.20	1200

The calculations for the field welds are shown in Fig. 27*B*. Some designers calculate these welds as being only in direct shear but this method does not take account of the eccentricity of the load $W/2$ with respect to the weld. From the sketch in the upper left-hand corner it will be seen that when the load is applied along the center line of the beam web the angles tend to twist, pressing together at the top and separating at their lower ends. The mutual reaction at the top may be designated by the force, P. Its point of application is indeterminate but a reasonable assumption would be a distance from the top of the angles equal to $^1/_{10}$ of A, the length of the individual welds. Assuming a triangular

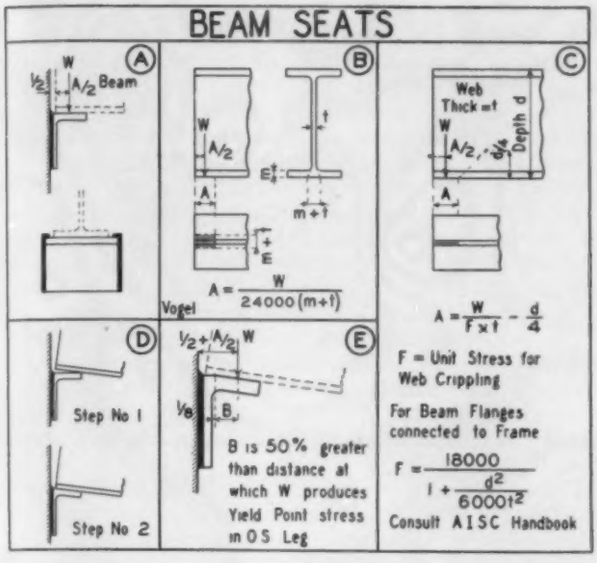


Fig. 29

distribution of horizontal stress on the weld, the resultant horizontal force must equal P and its point of application is $^2/_3$ of $0.9A$ below the upper force P. The value of P can be found by equating the moment of the horizontal couple to the moment of the load. The intensity of the horizontal stress, H, per inch is readily calculated when P has been found. The vertical stress per inch equals the load divided by the weld length. The resultant stress, R, is found by combining the vertical and horizontal stresses.

When beams frame opposite on a girder beam it is necessary to see that the web is of sufficient thickness to carry the shearing stresses. Reference should be made to the table given above.

It is evident that trial calculations to solve this problem would involve a very considerable amount of time and labor. The diagram of Fig. 28 was prepared to facilitate the operation. By reference to the sketch it will be seen that the field welds are carried over the top edge of the angles for $^1/_2$ in. The writer believes this to be a desirable precaution. This additional length is not included in the calculations.

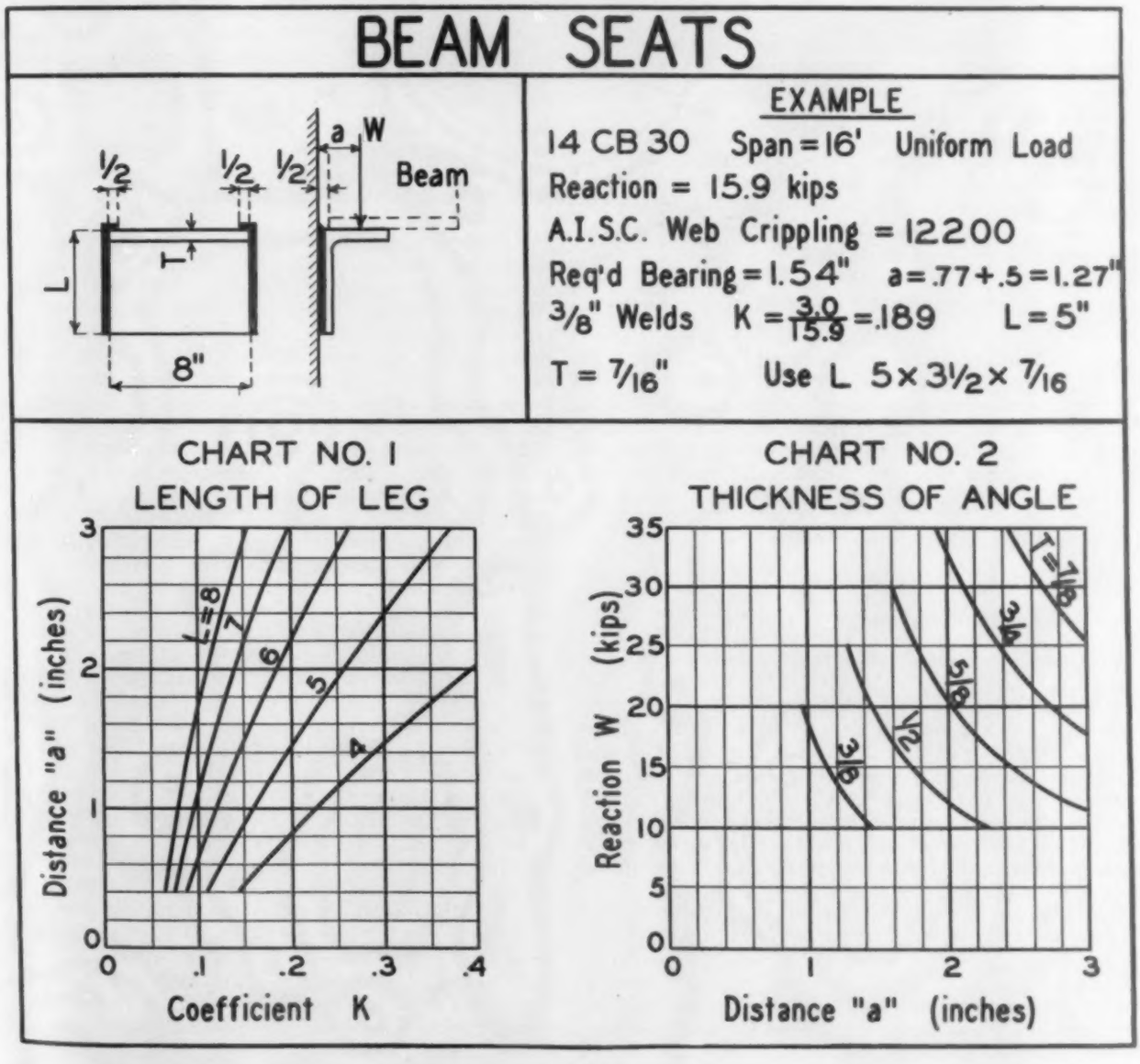


Fig. 30

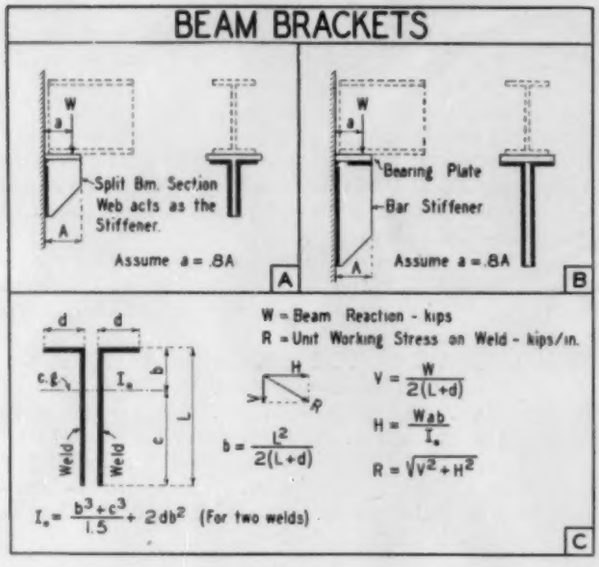


Fig. 31

The use of the charts can be illustrated with a typical example. Assume a 21 CB 58 beam with a reaction of 40 kips. The web thickness of this beam is 0.350 in. Entering the right-hand chart at the vertical ordinate for 40 kips, move horizontally to the diagonal line for web thickness, $t = 0.35$ in. The horizontal ordinate is found to be about $10^1/_2$ in., which is the required length, L, for the shop welds. The intersection falls within the range of $^5/_{16}$-in. welds so that this size may be used. The length of the field welds may be found in a similar manner from the left-hand chart.

A = 10 in. for $^3/_8$ welds or 11 in. for $^5/_{16}$ welds

If two such beams frame opposite, for example, on a 27 CB 85 girder beam, which has a web thickness of 0.450 in., the allowable stress on the weld is $G \times t/2 = 12{,}000 \times 0.450/2 = 2700$ lb. per in. This is less than the working value of a $^3/_8$-in. weld, hence, the length of weld should be calculated for the next smaller size of weld, i.e., $^5/_{16}$ in. which has a working unit stress of 2500 lb. per in.

The final design lengths are then:

Shop welds $^5/_{16}$ in., min. $L = 10^1/_2$ in.

Field welds $^5/_{16}$ in., min. $A = 11$ in.

A moment's thought will show that an unlimited number of lengths of angles might result from the calculations for each individual beam on a whole job. The writer suggests standard lengths and sizes as shown in the upper right-hand corner of Fig. 28. Since the girder beams have to be punched it is also well to arrange the holes on standard gage lines in order to facilitate fabrication. The first and second rows of holes from the top take the bolts which prevent bending stress coming on the welds while the lower holes in the longer angles permit of using bolts to draw the angles to the web for welding.

With the use of these standard angles, the final connection for the 21 CB 58 beam would be 2 Ls 3 x 3 x $^3/_8$ x 1 ft. 0 in. with at least $^5/_{16}$-in. welds. The weld lengths are $A = 11$ in. and $L = 12$ in., and must remain the same even if $^3/_8$-in. welds are used. For further simplification A can be 12 in. so that the welds extend for the entire vertical distance.

The next type of connection which will be considered is the so-called seat angle. This is a familiar connection in riveted construction and is one of the simplest and most satisfactory methods in welded construction whenever it can be used. Figure 29*A* shows a typical connection of this type. The problem is to select a point of application for the beam reaction, W, so that the welds may be calculated according to the method give in Fig. 12.

In an article on welded construction, Mr. Andrew Vogel of the General Electric Co. suggested the solution for the point of application shown in Fig. 29*B*. The load was assumed as distributed over the shaded area at a bearing of 24,000 lb. per sq. in. and the resultant was located at a distance $A/2$ from the end of the beam.

The web crippling formula of the American Institute of Steel Construction offers the most logical solution and is clearly set forth in Fig. 29*C*. The values of F are given for each beam in the A. I. S. C. Handbook, which makes the computation of the distance A a very simple matter.

The action of a beam seat may be divided into the two steps of Fig. 29*D*. In the first step the beam deflects under load and takes a slope at its support. This tends to throw all the reaction on the toe of the seat angle. In the second step the outstanding leg of the angle has deflected until at some point it has attained the same slope as the end of the beam. The location of the point at which the load is then acting will depend upon the thickness of the angle. The thicker the angle, the farther out on the leg the load must act in order to bend it to the required slope. The next step in the problem becomes the determination of the thickness which will keep the load at or within the distance $A/2$ from the end of the beam, as found by the A. I. S. C. method.

Calculations readily show that the outstanding leg is stressed in bending to the yield point in being brought to the required slope. Let d = distance at which the load W produces yield-point stress. From the theory of plastic bending at a distance of $3d/2$ distortion will be continuous without increase of load. In Fig. 29*E* the distance $B = 3d/2$. Evidently then, if the thickness is made such that the yield stress (assumed as 33,000 lb. per sq. in.) is attained at a distance, d, from the load, it will be impossible for the point of application of the load to exceed the distance, B. The section from which B is measured has been arbitrarily taken as $^1/_8$ in. from the inside face of the vertical leg. By equating the two distances to the load, as shown in the figure, it becomes possible to arrive at a solution for the thickness.

The solution becomes a simple matter by the use of the chart in Fig. 30. For most beams under which a seat angle is practical, a length of angle of 8 in. is great enough to provide space for field-welding the beam flanges to the seat. This length has, therefore, been used in the preparation of the chart. Again it will be noticed that the welding has been carried over the top of the angle for a short distance. This is the point at which the bending stress on the welds is greatest and it is well to add this extra strength, although it has not been used in the calculations. The lengths of leg given in Chart No. 1 are the common widths of angles. The example in the figure is sufficient to illustrate the use of the charts. The coefficient, K, is found by dividing the working stress on the weld by the beam reaction. For $^3/_8$-in. welds, at 3.0 kips per in., $K = 3.0/15.9 = 0.189$. From Chart No. 1, with $a = 1.27$ in., $L = 5$ in. This leads to the selection of a 5 x $3^1/_2$ angle. From Chart No. 2, with $W = 15.9$ kips and $a = 1.27$ in., the thickness is found to be about $^7/_{16}$ in. The final design is a 5 x $3^1/_2$ x $^7/_{16}$ angle, 8 in. long.

The use of seat angles should probably be limited to a

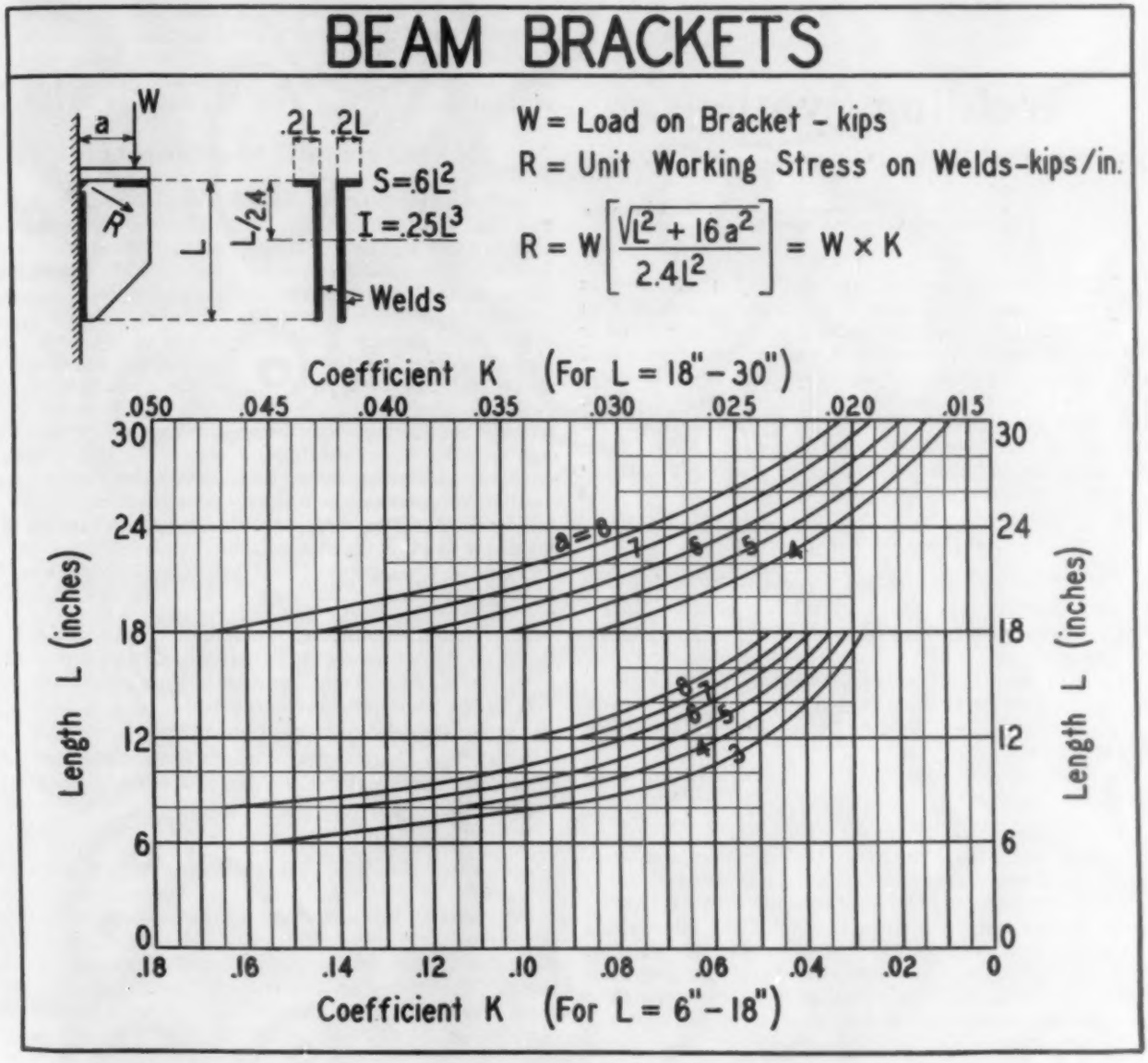

BEAM BRACKETS
W
a
.2L .2L
S=.6L²
I=.25L³
L/2.4
L
R
Welds
W = Load on Bracket - kips
R = Unit Working Stress on Welds-kips/in.
R = W [√(L² + 16a²) / 2.4L²] = W × K
Coefficient K (For L = 18"-30")
.050 .045 .040 .035 .030 .025 .020 .015
30 24 18 12 6 0
a=8 7 6 5 4
Length L (inches)
.18 .16 .14 .12 .10 .08 .06 .04 .02 0
Coefficient K (For L = 6"-18")
8 7 6 5 4 3

Fig. 32

maximum load of 35 kips and for beams not exceeding 18 or 20 in. in depth. One important point should be emphasized, namely, that if any horizontal force occurs which tends to pull the seat angle away from its support, then additional welding must be added across the top of the angle to resist this force.

When the loads exceed the capacity of the "seat angle" connection it is general practice to use the stiffened seats or brackets shown in Figs. 31*A* and *B*. The first type uses a split-beam section in which the web of the beam acts as the stiffener under the flange. The second type is built up from two pieces—a bearing plate and a bar stiffener. Calculations for the welding are given in Fig. 31*C*. The point of application of the load may be assumed as being at the distance $a = 0.8A$ from the face of the support. In order to have a proper amount of welding along the vertical edges of the stiffener it is recommended that in the design calculations the distance d shall not exceed 20% of the length, L. When $d = 0.2L$ the moment of inertia of the two welds about the center of gravity is $I_0 = L^3/4$ and the section modulus is $S = 0.6L^2$. Figure 32 is a chart to facilitate the design of such brackets and its use is identical with that of the chart in Fig. 12.

Conclusion

There are many problems upon which the writer, from necessity, has not touched. Time and considerations of length of the paper have set limitations. The general principles have been discussed and designing methods have been given which will apply to the problems arising in ordinary structural welding.

Welding Symbols

By JAMES W. OWENS*

✦ Mr. Owens is Chairman, A. W. S. Nomenclature Definitions and Symbols Committee.

ELSEWHERE in this issue of the Journal there is published the Society's Approved Arc and Gas Welding Symbols for Buildings, Bridges and Other Framed Structures. For the guidance of engineers and draftsmen using these and other Symbols of the Society, it seemed advisable to prepare a brief article giving a list of the Society's Symbols for use on drawings, and an outline of the fundamental principles on which these Symbols are being developed by the Society's Nomenclature, Definitions and Symbols Committee.

Symbols of the Society

The Symbols of the Society and their present status are as follows:

1. Arc and Gas Welding Symbols for Marine Construction.
2. Arc and Gas Welding Symbols for Buildings, Bridges and Other Framed Structures.
3. Arc and Gas Welding Symbols for Machinery Construction.
4. Resistance Welding Symbols for All Types of Construction.

The Arc and Gas Welding Symbols for Marine Construction are those published in the Nomenclature Bulletin of the Society in 1929. As the use of these symbols has indicated the need of some minor revisions, these revisions are now being made and the latest draft of these Symbols will shortly be published in the Journal. It is interesting to note that these Symbols have been found practical and economical and, as a result, they have not only been approved by the Bureau of Construction and Repair of the Navy, the Bureau of Engineering of the Navy and the American Marine Standards Committee, but are in general use in all navy-yards and naval stations and the major merchant shipyards of the country. They are at the present time undoubtedly the most widely used welding Symbols.

The Arc and Gas Welding Symbols for Buildings, Bridges and Other Framed Structures are those published in this issue of the Journal. They have not as yet been used as widely as the Marine Symbols, nevertheless, they have been found to be practical by the American Bridge Company and the General Electric Company. These Symbols will probably need some amplification on account of the variety of structural work on which they will be used and, should this be found necessary, it is recommended that reference be made to the Society's Marine Symbols and the fundamental principles outlined herein.

The Arc and Gas Welding Symbols for Machinery Construction have not as yet been prepared and, until the Nomenclature Committee is able to present a set of Symbols for the class of work, it is recommended that machinery manufacturers use either of the foregoing Symbols, or a combination of the two.

* Consulting Engineer and Director, National Weld Testing Bureau of the Pittsburgh Testing Laboratory, Pittsburgh, Pa.

The Resistance Welding Symbols for All Types of Construction, which were published in the Society's Nomenclature Bulletin in 1929, are now being revised, and it is hoped to publish the revised Symbols in the near future.

Fundamental Requirements

In the development of the Society's Symbols it has not been possible to rigidly adhere to each fundamental requirement for indicating welds on drawings as outlined in this article, however, these requirements are given so that a draftsman using the Symbols will not arbitrarily develop a prejudice against their use, or attempt to develop a set of Symbols of his own. On the other hand, if these requirements are studied, the draftsman can materially aid in the evolution of the Symbols along the lines on which the Nomenclature Committee has been working, so that we will ultimately not only have well-established standards but a set of Symbols which experience would have shown to be practical and economical in drafting-room practice.

The fundamental requirements for Symbols indicating welds on drawings are as follows:

(*a*) Should be simple, few in number and easily memorized.

(*b*) Should, if possible, eliminate the use of abbreviations, as the abbreviations used would not necessarily be the same in different languages, thus hampering the adoption of an international standard.

(*c*) Should readily be made free hand.

(*d*) When the drawing is made in ink, the Symbols should dry quickly to prevent the possibility of smearing.

(*e*) Should be readily erased if inked in.

(*f*) Should be readily painted on molds and patterns.

(*g*) Should not conflict with marks now used by piece-work counters.

(*h*) Should be fundamentally based on the *location* of the weld, the reinforcement or the bevel, rather than the form (shape) of the weld, the reinforcement or the bevel. The reason for this is that the fundamental and most generally used "forms" (fillet and butt) are obvious on all drawings, whereas "location" is usually not obvious.

The first Symbols developed in the United States in 1918 and the Symbols developed in foreign countries are based on the form of the weld. This method clutters up a drawing and serves no useful purpose. It is attractive to draftsmen who are novices in making drawings showing welding but useless to experienced draftsmen.

(*i*) Should be capable of being used *either directly on joint*, when this is considered necessary for clarity (particularly in detail drawings and for intermittent fillet welds) or *off the joint* by the use of an index line to the plane of application.

This latter method, when applicable, saves time and makes simple drawings possible. It is now used almost exclusively by the Bureau of Construction and Repair of the Navy and the major merchant shipyards, and the writer recently noticed that it has been adopted in Belgium.

(*j*) The fundamental principles used in the development of the Symbols should be capable of universal application in indicating welds made by the most generally used welding processes (gas, arc and resistance) and in indicating welds on drawings of all types of structures and products.

Abbreviations and Symbols for Use on Drawings of Buildings, Bridges and Other Framed Structures

Introduction

ABOUT two years ago, a Sub-Committee consisting of Messrs. H. M. Priest, Chairman, and A. Vogel were appointed to prepare a set of Symbols for Structural Work. The Symbols, abbreviations and a legend as developed by this Committee are now presented to industry as a tentative standard for use on drawings of buildings, bridges and other framed structures.

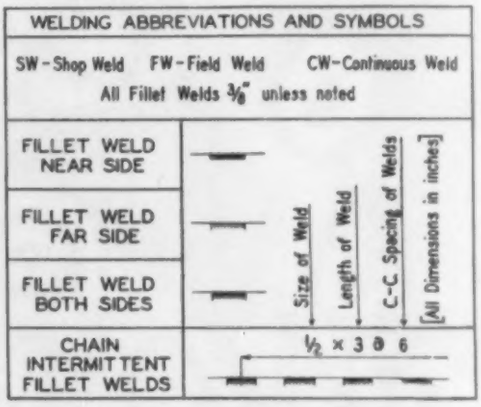
The Symbols for fillet welds are in principle fundamentally identical to the Society's Marine Symbols, which are now widely used, in that the *location* of the weld—near side, far side and both sides—is the determining feature. As butt welds are, at present, not extensively used in the class of work for which these Symbols are intended, the Committee felt that it would be preferable to show a section of the weld in all cases.

It is intended that a small size rubber stamp be made of the legend so that the Symbols can be stamped on all drawings on which the Symbols are used.

The Nomenclature, Definitions and Symbols Committee desires to express its appreciation to Messrs. H. M. Priest and A. Vogel for their work and to Mr. H. H. Moss of The Linde Air Products Company for valuable suggestions.

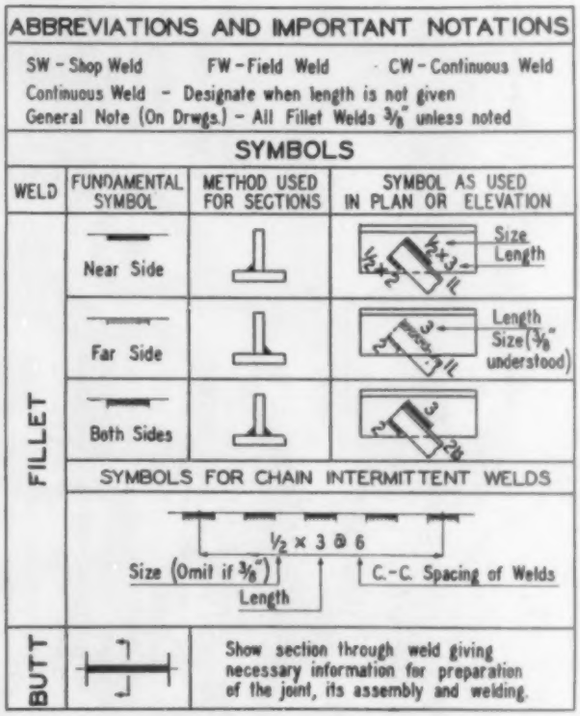
Members of Committee on Nomenclature, Definitions and Symbols.

JAMES W. OWENS, Pittsburgh Testing Laboratory, *Chairman*
L. C. BIBBER, Navy Dept.
W. S. BROBERG, Ord. Dept., U. S. A.
H. W. CRAVER, Engineering Societies Library
J. J. CROWE, Air Reduction Co.
J. H. DEPPELER, Metal & Thermit Corp.
F. M. FARMER, Electrical Testing Labs.
C. S. GILLETTE, Navy Dept.
F. T. LLEWELLYN, U. S. Steel Corp.
H. H. MOSS, The Linde Air Products Co.
J. G. RITTER, Westinghouse Elec. & Mfg. Co.
H. W. SAMSON, General Electric Co.
W. SPRAGAGEN, Sec., American Bureau of Welding
A. VOGEL, General Electric Co.
H. A. WOOPTER, Swift Electric Welder Co.


WELDING ABBREVIATIONS AND SYMBOLS
SW - Shop Weld FW - Field Weld CW - Continuous Weld
All Fillet Welds 3/8" unless noted
FILLET WELD NEAR SIDE
FILLET WELD FAR SIDE
FILLET WELD BOTH SIDES
CHAIN INTERMITTENT FILLET WELDS
Size of Weld
Length of Weld
C-C Spacing of Welds
[All Dimensions in inches]
1/2 x 3 @ 6

LEGEND FOR DRAWINGS
OF
BUILDINGS, BRIDGES AND OTHER FRAMED STRUCTURES
(ARC AND GAS WELDING)

Approved by American Welding Society Nomenclature Committee June 1933
Executive Committee June 1933


ABBREVIATIONS AND IMPORTANT NOTATIONS
SW - Shop Weld FW - Field Weld CW - Continuous Weld
Continuous Weld - Designate when length is not given
General Note (On Drwgs.) - All Fillet Welds 3/8" unless noted
SYMBOLS
WELD
FUNDAMENTAL SYMBOL
METHOD USED FOR SECTIONS
SYMBOL AS USED IN PLAN OR ELEVATION
FILLET
Near Side
Far Side
Both Sides
Size
Length
Length
Size (3/8" understood)
SYMBOLS FOR CHAIN INTERMITTENT WELDS
1/2 x 3 @ 6
Size (Omit if 3/8")
Length
C.-C. Spacing of Welds
BUTT
Show section through weld giving necessary information for preparation of the joint, its assembly and welding.

ABBREVIATIONS AND SYMBOLS
FOR USE ON DRAWINGS OF
BUILDINGS, BRIDGES AND OTHER FRAMED STRUCTURES
(ARC AND GAS WELDING)

Approved by American Welding Society Nomenclature Committee June 1933
Executive Committee June 1933

Welding and Cutting Nomenclature, Definitions and Symbols

Complete Report of 48 pages available at 25c per copy

TENTATIVE SPECIFICATIONS FILLER METAL

Report to the American Bureau of Welding of the Committee on Filler Metal Specifications

We are submitting herewith Revised Specification on Filler Metal (Electrodes and Welding Rods). The title of these specifications has been changed from "Welding Wire Specifications" to "Filler Metal Specifications" to conform to the definition for this material in the A. W. S. Bulletin on "Welding and Cutting Nomenclature, Definitions and Symbols."

The new filler metal specifications differ so radically from the old welding wire specifications that a brief explanation seems desirable. The old specifications referred largely to the composition and physical properties of the welding wire itself. Recent advances in the art and multiplication of the means for obtaining superior welds make the old system of specification no longer possible. This fact coupled with the recent trend in various codes to specify by ultimate result rather than intermediate means has led the Committee to adopt that method.

The revised specifications are based entirely on the properties of the joint and/or weld obtainable with the filler metal. They adhere to the latest standardized procedures for the various mechanical tests recommended for welds as covered by the report of the A. W. S. Committee on "Standard Tests for Welds" and are in harmony with other codes.

Scope of Specifications

These specifications cover Iron and Steel Electrodes and Welding Rods for use in the fusion welding of steel shapes, plates, bars, forgings, castings and other steel products covered by A. S. T. M. Specifications for material to be welded by the Fusion Process.

Respectfully submitted,

WELDING WIRE SPECIFICATIONS COMMITTEE

C. A. McCUNE, *Chairman*

L. C. BIBBER	F. C. ELDER	E. F. LUNDEEN
V. J. CHAPMAN	J. J. FLAHERTY	D. L. MATHIAS
J. L. CHRISTIE	C. F. GAILOR	W. B. MILLER
*A. CHURCHWARD	A. E. GAYNOR	J. W. OWENS
J. J. CROWE	*J. B. GREEN	J. J. SHUMAN
J. H. DEPPELER	C. J. HOLSLAG	W. C. SWIFT
	J. C. LINCOLN	E. WANAMAKER

* Deceased

SECTION I
AMERICAN WELDING SOCIETY SPECIFICATIONS FOR FILLER METAL (ELECTRODES AND WELDING RODS)

IRON AND STEEL

Issued April 1, 1920
Revised May 1, 1921
Revised June 1, 1933

1. *General:*

The following specifications are recommended for iron and steel filler metal (Electrodes and Welding Rods) for use in fusion welding. These specifications are primarily intended for filler metal up to $^5/_{16}$ inch diameter, inclusive, and tested on base metal up to $^1/_2$ inch thick, inclusive. The specifications are based on the physical properties of the weld produced by the filler metal and not on its chemical composition before use.

2. *Scope:*

These specifications are recommended as covering the requirements of the American Welding Society Code for Fusion Welding and Gas Cutting in Building Construction, the A. W. S. Rules for the Construction of Marine Boilers and Pressure Vessels, the A. S. M. E. Boiler Code, and the usual railroad, shipyard and industrial requirements as allowed by authoritative regulating bodies, such as the American Bureau of Shipping, the U. S. Steamboat Inspection Service, the Interstate Commerce Commission, etc.

3. *Material:*

Filler metal may be made by any process that will fulfill the requirements of these specifications.

4. *Physical Characteristics:*

Filler metal shall be made of commercially straight wire of uniform quality free from harmful segregation, oxides, pipe, seams or other irregularities. Wire diameter shall not vary more than plus or minus 3 per cent from that specified. Cut lengths shall not vary more than $^1/_8$ inch plus or minus from that specified. The surface of the wire shall be smooth and free from harmful scale, oil or grease and may be plain or copper finished as specified. If coated or covered, the coatings and coverings shall not be readily removable by ordinary handling and shall be of commercially uniform thickness and shall present a workmanlike appearance.

5. *Recommended Sizes of Wire and Rods:*

Minimum Deposited Metal Require-

J. C. LINCOLN

* Deceased

uniform thickness and shall present a workmanlike appearance.

5. *Recommended Sizes of Wire and Rods:*
$^3/_{32}$, $^1/_8$, $^5/_{32}$, $^3/_{16}$, $^1/_4$, $^5/_{16}$ inch diameter.

6. *Uses:*

For welding of steel shapes, plates, bars, forgings, castings and other steel products covered by A. S. T. M. Specifications for material to be welded by the Fusion Process.

7. *Tests:*

In the hands of a qualified welding operator,†† filler metal shall flow smoothly and evenly without any harmful characteristics. The welded joint and the deposited metal produced shall have the following properties:

Minimum Joint Requirements

Specification Number*	Welding Positions Required†	Base Metal	Form of Joint, Fig. 1	Treatment of Welded Specimen	Tensile Test, Ultimate Strength, Fig. 2	Free Bend Test, Elongation (Per Cent), Fig. 3	Nick-Break Test, Fig. 4
E-10 G-10	Flat	A. S. T. M. A-70 Firebox Quality 55,000 Lb. per Sq. In. Min. or Equivalent or A. S. T. M. A-114 Class A Marine Boiler Plate 60,000 Lb. per Sq. In. Min.	Single or Double V 60° to 90° Included Angle	**Stress Relieved	100% of Min. Base Metal Requirements	30%	Not Required
E-20 E-20 GP G-20 G-20 GP	Flat F. V. & O. Flat F. V. & O.	A. S. T. M. A-70 Firebox Quality 55,000 Lb. per Sq In. Min. or Equivalent	Single or Double V 60° to 90° Included Angle	**Stress Relieved	95% of Min. Base Metal Requirements	20%	Not Required
E-30 GP G-30 GP	F. V. & O.	A. S. T. M. A-70 Firebox Quality 55,000 Lb. per Sq. In. Min. or Equivalent	Single or Double V 60° to 90° Included Angle	None	85% of Min. Base Metal Requirements	10%	Required
E-40 GP G-40 GP	F. V. & O.	A. S. T. M. A-140 Structural Quality 60,000 Lb. per Sq. In. Min.	Single or Double V 60° to 90° Included Angle	None	90% of Min. Base Metal Requirements	7%	Required

* E denotes Welding Electrodes.
G denotes Welding Rods.
GP General Purpose.

** Stress relieving shall be within the range of 1100° to 1200° F. for $^1/_2$ hour for $^1/_2$ inch of thickness, based on 1 hour per 1 inch of thickness.

† F—Flat V—Vertical O—Overhead

†† For the requirements of a qualified welding operator, see American Bureau of Welding Bulletin No. I, "Standard Tests for Welds."

Minimum Deposited Metal Requirements as Measured on All Weld Metal Specimens

Specification Number*	Welding Positions Required†	Treatment of Specimen	Ultimate Strength, Per Cent of Base Metal Requirements, Fig. 6	Elongation, Per Cent in 2" Tensile Test, Fig. 6	Specific Gravity, Fig. 7
E-10 G-10	Flat	Stress Relieved**	100%	20.0	7.80
E-20 E-20 GP G-20 G-20 GP	Flat F. V. & O. Flat F. V. & O.	Stress Relieved**	95%	13.5	7.75
E-30 GP G-30 GP E-40 GP G-40 GP	NO TESTS REQUIRED				

*E denotes Welding Electrodes.
G denotes Welding Rods.
GP denotes General Purpose.

†F—Flat
V—Vertical
O—Overhead

**Stress relieving shall be within the range of 1100° to 1200° F. for $^1/_2$ hour for $^1/_2$ inch of thickness, based on 1 hour per 1 inch of thickness. Specimen to be stress relieved before machining to size.

8. *Inspection:*

All tests shall be made at the place of manufacture unless otherwise specified, and shall be at the expense of the purchaser.

9. *Packing:*

It is recommended that filler metal shall be delivered as specified in coils or in straight, 14- or 18-inch lengths for electrodes, and 36-inch lengths for welding rods, packed, wrapped and suitably protected as follows:

(*a*) Bundles of 50 lb. net weight.
(*b*) Boxes or kegs of 50, 100, 200 or 300 lb. net weight.
(*c*) Coils or reels of approximately 50, 100 to 200 lb. net weight.

10. *Markings:*

All bundles, boxes, kegs, coils or reels shall be suitably marked and bear the following information which shall be available at destination:

Make.....
Specification No.....
Diameter.....
Net Weight.....

11. *Ordering:*

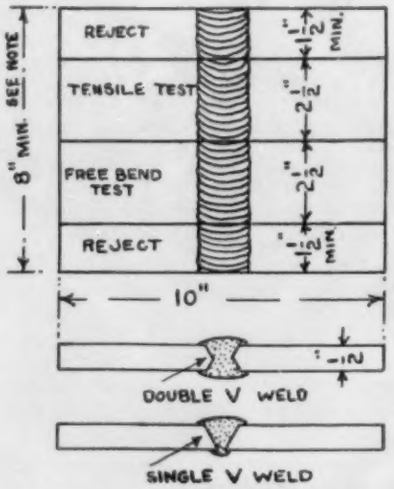
Material ordered under these specifications shall be designated as: "Filler Metal, Iron and Steel," American Welding Society Tentative Specifications revised June 1, 1933.

All orders shall be specified in pounds, including coating, if any. In addition, requisitions shall show the following:

Specification No.....
Size.....

PHYSICAL TESTS

WELDED JOINT SPECIMENS



Test plates shall be so supported that warping due to welding shall not throw the finished test plate out of line by an angle of over 5°.

If plates are warped, they shall be straightened cold before being stress relieved where required.

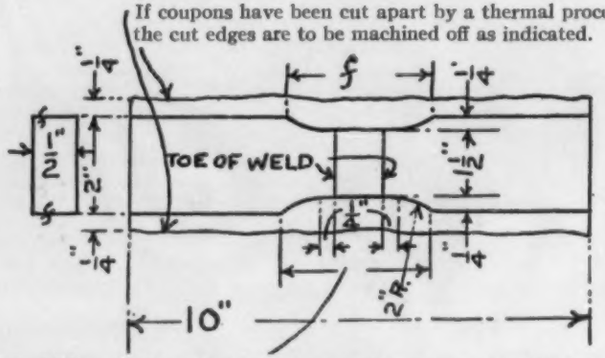
Plates may be of such dimensions that all specimens may be made from same plate if desired.

Test plates shall be welded under the procedure recommended by the manufacturer of the filler metal. For single V welds a reinforcement or wash weld is permitted on under side. Reinforcements shall not be more than 1/8 inch above surface of plate.

Both surfaces of weld to be machined flush with the plate in all cases.

Fig. 1—Test Plate Dimensions and Specimen Locations for Joint Determinations.
3 Required for Each Welding Position Specified

NOTE: When nick-break test is required, plate length should be increased to $10^1/_2$" to provide for specimen.



Machining of this section should preferably be done by milling. Both faces of the weld are to be machined flush with the plate.

Fig. 2—Joint Specimen for Tensile Test.
3 Required for Each Welding Position Specified

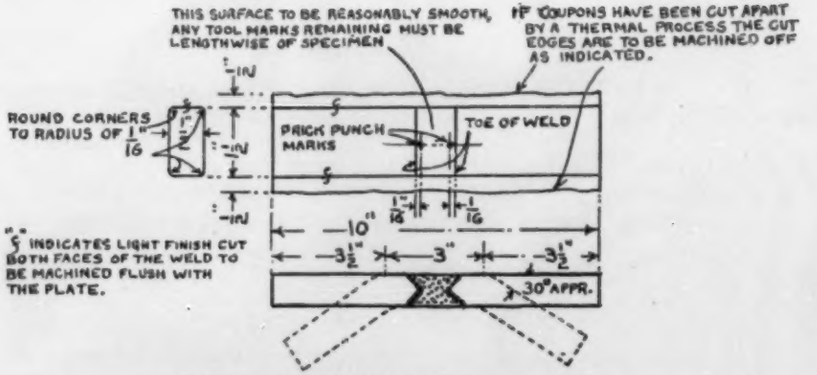

THIS SURFACE TO BE REASONABLY SMOOTH, ANY TOOL MARKS REMAINING MUST BE LENGTHWISE OF SPECIMEN
IF COUPONS HAVE BEEN CUT APART BY A THERMAL PROCESS THE CUT EDGES ARE TO BE MACHINED OFF AS INDICATED.
ROUND CORNERS TO RADIUS OF 1/16"
1" IN
½"
1" IN
PRICK PUNCH MARKS
TOE OF WELD
1/16"
1/16"
10"
3½"
3"
3½"
30° APPR.
f INDICATES LIGHT FINISH CUT BOTH FACES OF THE WELD TO BE MACHINED FLUSH WITH THE PLATE.

Fig. 3—Joint Specimen for Free Bend Test.
3 Required for Each Welding Position Specified

Bend-Test Specimen. The dimensions of the specimen shall be in accordance with Fig. 3. The length may vary with the thickness of the piece and is unimportant, provided it is long enough to permit the bending operation. The bend in the specimen may be started by holding the specimen in a vise about one-third from the end, producing an initial bend at this point with hammer blows. Another bend about one-third from the other end may be produced in the same manner.

The specimen with the initial bend at each end is then placed as a strut in a vise or compression machine and pressure applied gradually (that is, without shock) at the ends until failure occurs in the outside fibers of the bend specimen. When a crack is observed in the convex surface of the specimen between the edges, the specimen shall be considered to have failed and the test shall be stopped. Cracks at the corners of the specimen shall not be considered as a failure. The appearance of small defects in the convex surface shall not be considered as a failure if their greatest dimension does not exceed 1/16 inch.

The performance of the specimen is evaluated by measurement of the outside fiber elongation. This measurement may be made by means of a flexible scale. The elongation shall be recorded in per cent.

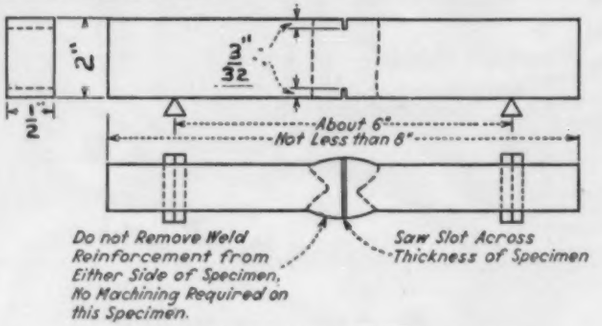

2"
3/32"
1½"
About 6"
Not Less than 6"
Do not Remove Weld Reinforcement from Either Side of Specimen, No Machining Required on this Specimen.
Saw Slot Across Thickness of Specimen

Fig. 4—Specimen for Nick-Break Test.
3 Required for Each Welding Position Specified

Nick-Break-Test Specimen. The shape and dimensions of the specimen shall be as shown in Fig. 4. The specimen is to be suitably supported as shown in Fig. 4 and broken by a sudden blow or blows applied at the center of the weld. The blow should

3 Required for Each Welding Position Specified

broken by a sudden blow or blows applied at the center of the weld. The blow should

preferably be applied by a power hammer or falling weight, and be of sufficient intensity to cause a sharp, sudden fracture of the specimen through the nicked portion.

Nick-Break Test. The nick-break test for soundness of the weld shall show in the fractured surface complete penetration through the entire thickness of the weld, absence of oxide or slag inclusions and a degree of porosity not to exceed six gas pockets per square inch of the total area of the weld surface exposed in the fracture, the maximum dimension of any such pocket not to be in excess of $^1/_{16}$ inch.

PHYSICAL TESTS
DEPOSITED METAL SPECIMENS

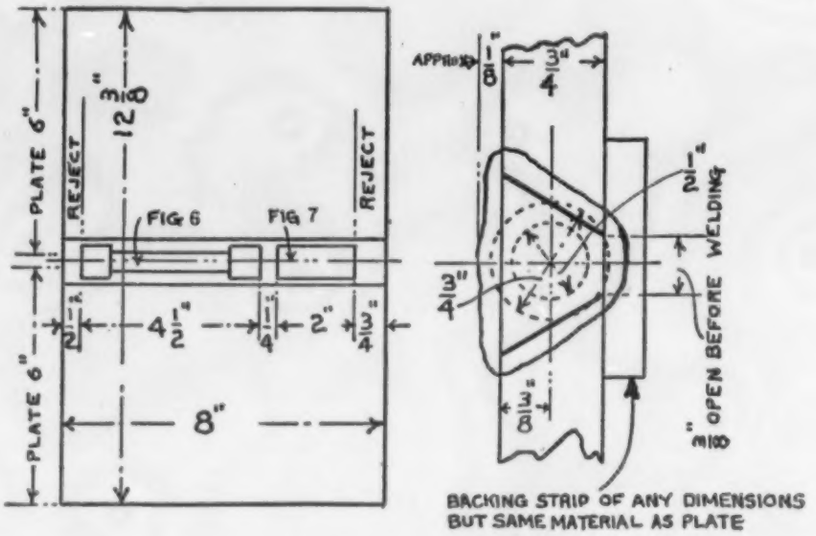
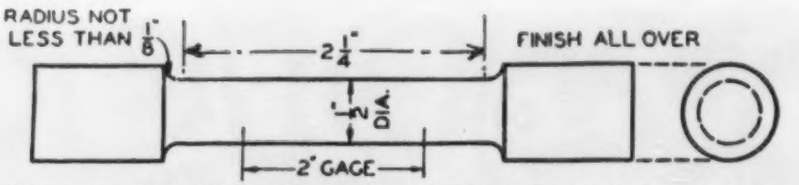


Fig. 5—Form for Depositing Metal for Tensile and Specific Gravity Tests

The shape and dimensions of the tensile and specific gravity test specimens are shown in Figs. 6 and 7. They shall be prepared from a bar of weld metal 8 inches long. This bar shall be formed entirely by depositing filler metal between two plates as shown in Fig. 5 employing the most suitable technique. For metal arc welding subsequent layers shall not be deposited until previously deposited layers are cold to the hand. Hammering may be done for the purpose of removing scale only. The base metal used for the form shall be of the same composition as that used in any corresponding joint tests. After the bar is formed, the base metal may be removed by a gas torch or any suitable machining operation. When gas cutting is employed, care should be taken that the cut is sufficiently distant from the weld metal to avoid any heat treatment effect on the weld metal. Carbon blocks may be used in lieu of plates for preparing gas-welded specimens.



The gage lengths, parallel portions and fillets shall be as shown, but the ends may be of any form which will fit the grips of the testing machine.

NOTE: A diam. of 0.505 in. may be used if desired. This gives an area of 0.200 sq. in.

Fig. 6—Deposited Metal Specimen for Tensile Test.
3 Required for Each Welding Position Specified

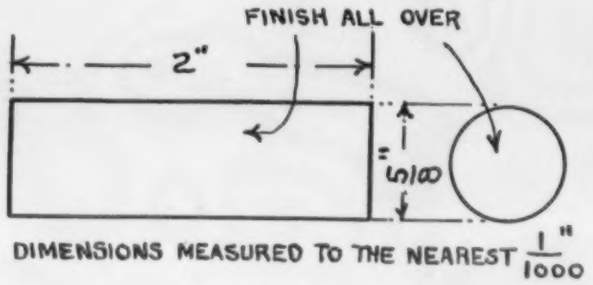


Fig. 7—Deposited Metal Specimen for Specific Gravity Test.
3 Required for Each Welding Position Specified

The specific gravity shall be determined by measurement and weight of the specimen and not by the immersion method.

Tests on the Fatigue Limit of Welded Joints Subjected to Repeated Tension Stresses

By PROF. OTTO GRAF

✦Prof. Graf is connected with the Technical College, Stuttgart, Germany.

DRAWINGS of welded steel structures, or of welded machinery, often showed only a few years back that these were constructed along the same lines as riveted constructions. Framework was welded in the same constructional manner. The girders were simply connected with fillet-welds instead of rivets; in rotating machine parts the hub and rim were joined as before with spokes which were originally riveted and then latterly connected with fillet-welds. When two years ago in Germany the necessity arose to fix the working stresses for welded structures, it was pointed out, by example of certain experiments, that the resistance of weldings subjected to repeated stresses must be the object of careful research. It needed to be specially emphasized that butt-welds and fillet-welds could not be regarded as equal in reliability.[1]

Fig. 1

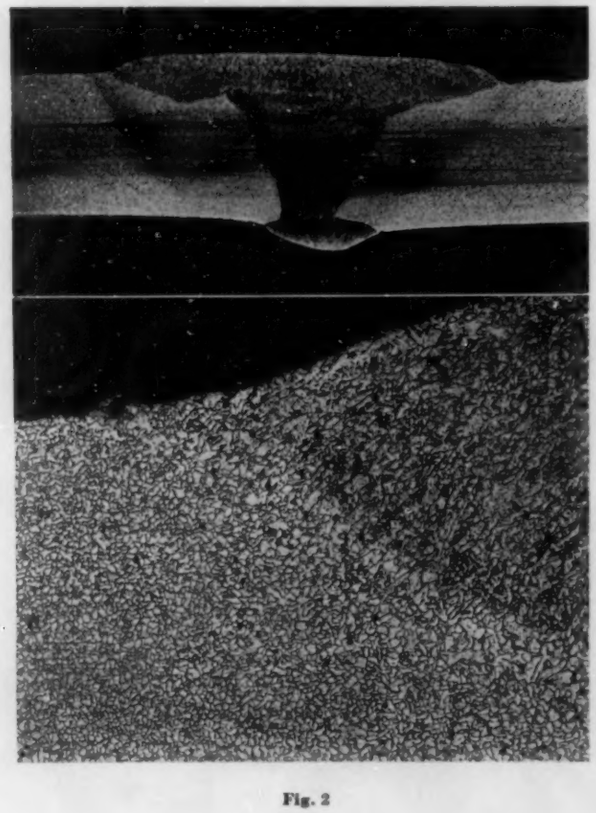


Fig. 2

Fig. 3

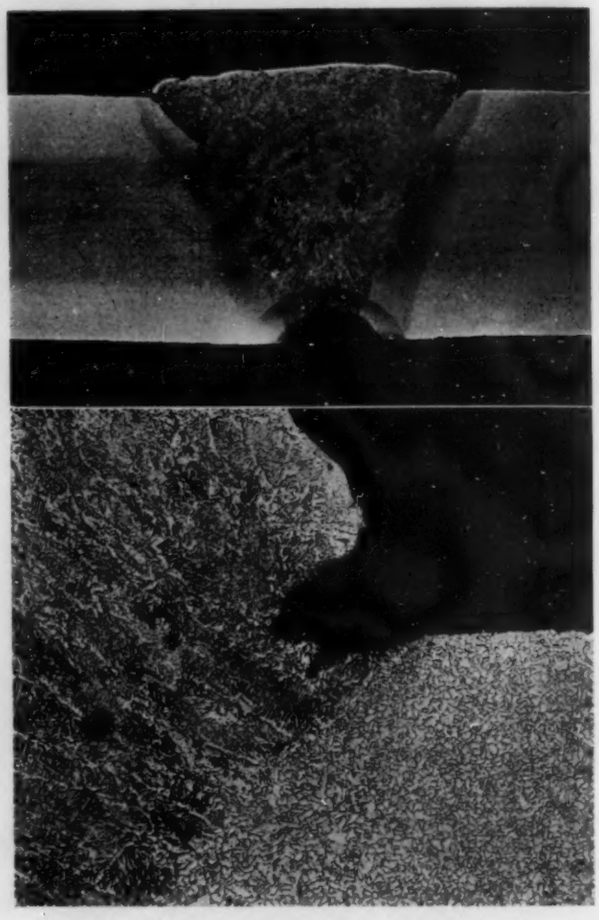


Fig. 4

This procedure recommended itself because the results of fatigue tests indicated that an essential change in methods was necessary, the then existing methods being totally derived from the results of static tension tests. Some examples of the work performed will illustrate the stand of the matter up to this point in respect to tension tests. Over bending and compression tests a further report will be given.

The first problem is: *What amount of resistance is shown at the decisive section in riveted joints as compared with weldings when the now existing specifications are followed?*

For these tests we used connections made of flat iron and channel bars. Here only joints made of flat iron are discussed. In the static tension tests the full tension strength was recorded for the riveted, as well as for the welded connections. In no case did the welds themselves fail. In respect to the loads carried the weldings showed themselves superior because the reduction caused by the bore-holes was obviated.

By subjecting the joints to repeated tension stresses, in such a way that the stresses rose from a small amount to a maximum, then again falling to the initial load at the rate of 350 times per minute, the above-stated aspect changed considerably.

[1] For reference on these statements see Graf, Dauerversuche an Stählen mit Walzhaut, ohne und mit Bohrung, von Niet- und Schweissverbindungen," Berlin, VDJ publication 1931, p. 38 and onward, further, *Stahlbau*, 1931, p. 258 and *Bautechnik*, 1932, p. 395.

Fig. 5

Unmachined butt-welds of good finish, of mild steel, gave a tension fatigue limit of 15 to 18 kg./mm.², joints with longitudinal fillet-welds only 6 to 14 kg./mm.², whereas in the latter case results of over 10 kg./mm.² only appeared in one series of tests.[2]

It was shown thereby that the efficiency of the materials in weldings is very varied and lies in the case of butt-welds much higher than with longitudinal fillet-welds, and moreover it only reaches with butt-welds the results obtained with riveted joints. These results were to be expected, but up to this point exact figures were not available.

Further investigations have been carried out on question 2: *How shall a butt-weld be designed, which can meet often repeated stresses with great resistance?*

It was found, as was to be expected by giving thought to the matter, that welds with few pores, with good finish on both sides and furthermore with gradual sectional changes give high fatigue limits for tension. The Figs. 1 and 2 show a satisfactory butt-weld with a fatigue limit with a minimum stress at zero of 18 kg./mm.² The weld had only a few pores, showing a gradual transition from the base metal to the welded part.

The contrary is illustrated in Figs. 3 and 4; the weld stands sharply out against the base metal showing many pores; the fatigue limit was in this case 10 kg./mm.².[3]

Whereas, especially in the case of unmachined welds with steeply inclined flanks, the resistance against repeated stresses is lowered, it is to be expected, in the case of welds with few pores, that this can be increased by machining the weld. The weld in Fig. 2 showed no improvement when machined.[4]

The resistance of butt-welds can furthermore be increased by special design. Bars with welds as per Fig. 5 gave a fatigue limit of 22 kg./mm.²; by such results is shown that the diagonal welds produced higher results.[5]

In general it can be assumed on the ground of the investigations made that it is possible to produce butt-welds with such high resistance, that the bearing strength under often repeated stresses is not smaller, but can become higher than that of riveted connections. In applying these facts it must be, of course, borne in mind that faults in butt-welds are of far greater effect than in riveted joints, not considering the fact that in the latter case faults will be very seldom and will become only gradually evident, so that they can be remedied in time. The relation between the fatigue strength and the working stress will have to be fixed differently for butt-welds than in the case of riveted joints except that the weld before coming into use is found to be faultless by exceptionally careful tests.

This line of thought always gives rise to the wish to use fillet-welds as far as possible instead of butt-welds because with fillet-welds faults are of less consequence than with butt-welds. In addition to this fillet-welds are often simpler and, therefore, cheaper than butt-welds.

Fig. 6A Fig. 6B

Fig. 7

[2] We know for comparison that flat iron bars of mild steel without bore-holes, but with rolled surface give data for the fatigue limit up to approximately 30 kg./mm.², whereas such bars with rolled surface but with bore-holes showed a fatigue limit of 16 up to 21 kg./mm.², i.e., an average of 18 kg./mm.²; with satisfactorily riveted joints the results obtained with flat bars with bore-holes are reached.

[3] With reference to the effect of other defects compare a report in *Stahlbau*, 1933.

[4] Compare also the dissertation of Pfeiffer, Stuttgart, 1929, p. 42.

[5] These results apply to gas-welds and have since been confirmed by Schulz with arc-welds.

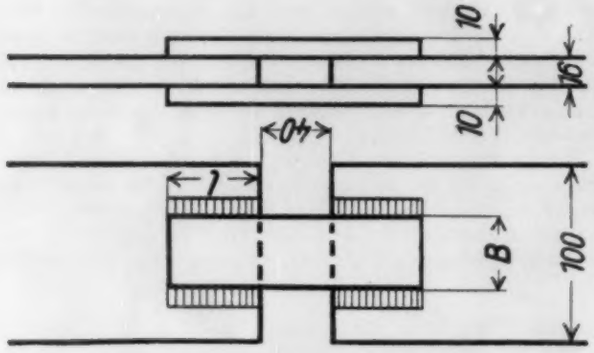


Fig. 8

The results obtained up to the present speak against this in the case where the stressing is wholly or to the greater part due to often repeated loads. The investigation of fillet-welds must, therefore, find special consideration and the present limits of their use must be extended by improvement of the materials and of the design. This is our third problem.

In the first place, let the *illustration of the fracture of joint with fillet-welds come under consideration.* The fracture starts as is to be expected at the end of the weld, compare Fig. 6, and then takes a slanting course to the direction of the tension stress, see Fig. 7. If in a connection with fillet-welds greater resistance is desired, the peak stresses must be diminished at the point of the commencement of the fracture. This can be effected in a small degree by reduction of the width of the connected bars. With specimens as per Fig. 8 was found:

Width B	25	40	70 mm.
Length	31	50	88 mm.
Fatigue limit D	10	9	7 kg./mm.[2]

Furthermore, the length of the connection is of importance. The strength can be raised up to a certain limit by increasing the length of the welding.

The form of the weld and also its thickness, when kept within the chosen limits, was not of special effect.

Problem 4 is also important: *How shall joints with fillet-welds at the butt end be formed if they are subjected to often repeated stresses?*

Figure 9 shows such joints after having been subjected to 2,094,500 cycles under tension stress at a range of 12 to 18 kg./mm.[2] The course of the fracture implies that satisfactory welding is necessary at the base of the weld and that the inclination of the flanks should be as gradual as possible. Further investigations on this question are being carried on.

Questions are often put as to the progress of our 5th problem: *Is it advantageous to provide butt-welds with cover plates? How should the cover plates be designed, if they are intended for weldings subjected to often repeated stresses?*

Many tests with plates of different design showed, in general, that plates connected with unmachined fillet-welds only give stronger joints when they have to support unsatisfactory butt-welds. The change of section at the ends of such cover plates (compare for example Fig. 10) is always of such account that satisfactory butt-welds prove to be of greater resistance when subjected to repeated stresses than welds with cover plates. Only when the plates were subsequently machined, so that the change of section was gradual, are such joints equal to connections with unmachined butt-welds.[6]

[6] For further information see *Stahlbau*, 1933.

The examples stated up to the present concern the fatique limit when the minimum stress was zero. For further evidence problem 6 was taken up: *In what degree does the range of stress vary which is borne by weldings subjected to tension stress when static loads are added?* For this effect joints with butt-welds, longitudinal fillet-welds and with fillet-welds at the butt ends were tested. It was found in accordance with that which was generally known, that the endurance range decreases with increase of the simultaneously acting static loads. It must be noted that the decrease kept within the limits shown by the tests with bars with bore-holes.[7]

Furthermore it could be even better seen than in the tests for Problem 1 that fillet-welds are, at present, finding good use when the stresses are chiefly caused by static loads.

With such data, which is continually being increased not only for tension tests but also by compression and flexure tests, the fundamentals are being evolved for the revision of the standard, specification DIN 4100 with regard to fixing the working stresses of weldings especially for parts subjected to repeated stresses. Beyond this experience is being gained for the selection of the electrodes and for a satisfactory design of weldings.

[7] For further information see *Stahlbau*, 1933.

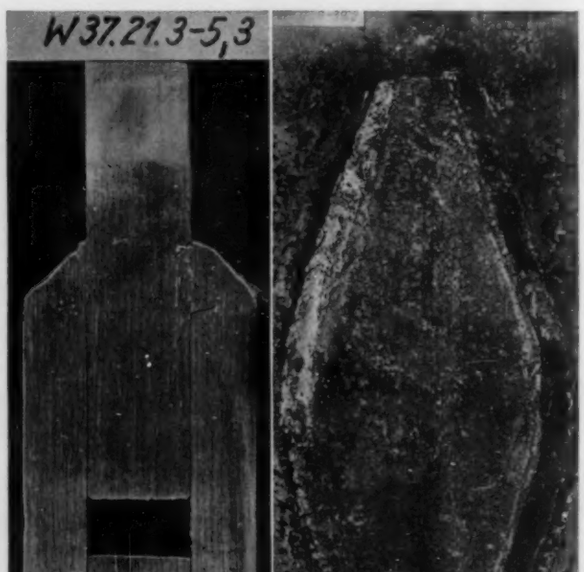


Fig. 9

Fig. 10

Discussion of Paper by Bela Ronay on "The Governing Factors of the Performance of Mild Steel Electrodes"

By GILBERT E. DOAN

✦Gilbert E. Doan is Associate Professor, Dept. of Metallurgical Engineering, Lehigh University.

THE paper by Bela Ronay in the May Journal, called "The Governing Factors of the Performance of Mild Steel Electrodes," is one of interest, and the author is to be congratulated upon a careful and profitable study of the depositing performance of electrodes during welding. The writer regrets, however, that more support is not given in the paper for some of the rather startling statements and conclusions presented. For instance, in the second paragraph, "It is believed that the Arcronograph . . . analyzes a weld with equal or greater reliability . . . than the X-ograph," seems rather a hasty conclusion on the basis of the data presented in the paper. Assuming that the meter does show the lengths of the periods of arcing and of short circuit, as well as the actual current and voltage (and, therefore, the actual arc length) during welding, these data would scarcely be sufficient to determine fully the soundness of the weld. For example, porosity in the weld due to gas content of the electrode and the rate of cooling of the weld metal might not be recorded on the chart, whereas a radiographic exposure would show porosity plainly. Again, shrinkage cracks in a portion of the weld often form after the arc has passed over that portion. These cracks would appear in a radiograph, but not on the current and voltage charts. The writer is unable to find anything in the paper which would indicate that the meter records would reveal these two principal defects in welds—porosity and shrinkage cracks.

The idea that non-metallic inclusions in the electrode cause explosions also seems not well supported. If these inclusions are silicates, for example, they would not necessarily explode at the temperature of the globule, even though it were well above the melting point of iron. Such inclusions exist quietly in a steel-making furnace and in the ladle at similar temperatures. They may dissociate somewhat, giving rise to a vapor pressure, it is true, but the iron itself probably forms quite a volume of vapor at the cathode spot.

Under sub-head "1. Chemical Composition," the statements are made that "The carbon content determines the melting point of the metal, thus regulating its viscosity. The surface tension of the molten metal at the tip of the electrode is proportional to the viscosity . . ." Unfortunately, the relationships between melting point, viscosity and surface tension have never been established and in many cases these properties seem to be entirely unrelated. For example, mercury has a fairly high surface tension, causing it to form globules when spread out on a clean glass surface, but both its melting point and viscosity are low. Molasses, on the contrary, is quite viscous and it flows slowly at room temperature, but its surface tension is low, so that it "searches" and penetrates into fine cracks if allowed sufficient time.

Under sub-head "3. Grain Structure," it should perhaps be pointed out that the specific resistance of steel changes very little with the grain size, certainly less than 3 per cent.

Under sub-head "4. Surface Finish," the higher current densities used with covered electrodes might conceivably lead not to a higher temperature of the globules as is assumed in the paper, but merely to a more rapid melting and detachment thereof.

In conclusion, the writer wishes again to commend the author's studies in the belief that they have yielded an important method for the selection of electrodes, where information is certainly needed. In so far as the method of recording instantaneous current and voltage of the arc is cheaper, more portable and quicker than the oscillograph, it constitutes a valuable development in the study of deposition characteristics of electrodes.

Internal Combustion Engine Cylinder Head Manufactured by the Electric Arc Welding Process

By E. WANAMAKER

✦Mr. Wanamaker is Electrical Engineer of the Chicago Rock Island and Pacific Railway Company.

WITHIN the last few years there have been introduced in rail service quite a large number of motor cars and locomotives, the prime movers of which are internal combustion engines. Most of these engines range in size from 200 to 400 H.P. capacity, with a bore and stroke ranging from 7 x 8 in. to 8 x 10 in., the speed varying in most cases between 1100 and 900 r.p.m., depending upon the size. There have, however, been built some small engines and also some larger engines.

Practically all of these internal combustion engines have been built with cast-iron bases, lower and upper crank cases, cylinder water boxes or cylinder blocks and cylinder heads.

Some of the engines were placed in service where their full power capacity was required for a very large percentage of the time they were on the run. In such cases the cylinder heads were necessarily subjected to much greater thermal stresses and the resultant strains, in many instances causing failure of the heads due to cracking, thus occasioning very high and excessive maintenance expense. Also, the wear or erosion of the valve seats was sufficiently great to materially shorten the life

of the heads even though they did not crack. These heads were of the "One head common to two cylinders" type.

To overcome the shortness of the life, occasioned by excessive wear and erosion of the valve seats, an effort was made to utilize renewable valve seats, and thus prolong the life of the head. This method, however, was found to be unsatisfactory, as it was impossible to consistently keep the seats tight in the head, with the result that the loose seats would cause failure and excessive damage, and even had the renewable seats proved successful, the cracking of the heads would not have been prevented.

On one railroad having a large number of internal combustion engines and which experienced a great deal of trouble with the cast-iron heads, one of the welding fraternity conceived the idea of fabricating a head from sheet steel assembled by use of the electric arc welding process. Most of the designing engineers approached on the subject took the stand that such a head would not function satisfactorily. However, the Mechanical Department of that railroad decided in view of the necessity of securing lower cost, longer life and more satisfactory heads to encourage the development of a welded head, which was eventually successfully perfected.

Dies were made for forming the combustion chamber, the intake and exhaust passages, front and rear sides, and the top cover section—the other two sides and bottom section being made out of plates cut to shape.

One-fourth in. and $^{3}/_{16}$-in. plate metal was used in the fabrication of most of these heads.

The welding procedure was initiated with the welding of the intake and exhaust passages in the combustion chamber—next, welding rods of bar stock to the combustion chamber in their proper position for cylinder head studs and spark plug openings.

Next, bar stock rods were welded to the intake and exhaust passages, to be later drilled for valve guides.

The bottom section was then fitted around the lower edge of the combustion chamber wall, with holes through which the lower ends of the rods provided for cylinder head studs projected, these rods later being welded to the lower section at the same time as the lower section was welded to the lower edge of the combustion chamber wall.

Next, the top cover section, which is provided with holes to correspond to the cylinder head studs and the valve guide bushing rods, was placed in position with the rods projecting through the holes, and the rods welded in to the top cover section by filling in between the rod and the walls of the hole.

Following this operation, the back side, which contains openings for the exhaust and intake passages, and for the spark plugs, was placed in position, welding the intake and exhaust passages and spark plug rods to the side, then welding the side to the top and bottom sections.

Next, the front side, which is of irregular shape and contains holes for spark plug rods, was fitted into place, and the welding of the spark plug rods in the openings provided for same was completed, following which the side was welded to the top and bottom sections.

The other two sides, consisting of flat steel plates, were then inserted in position and welded to the front and back sections, and to the top and bottom sections.

Upon completion of the welding operation, the bottom, top and back are machined, followed by the drilling of the cylinder head stud rods—meaning that the holes for the cylinder head studs are formed by drilling a hole through the rods that were welded into the cylinder head assembly.

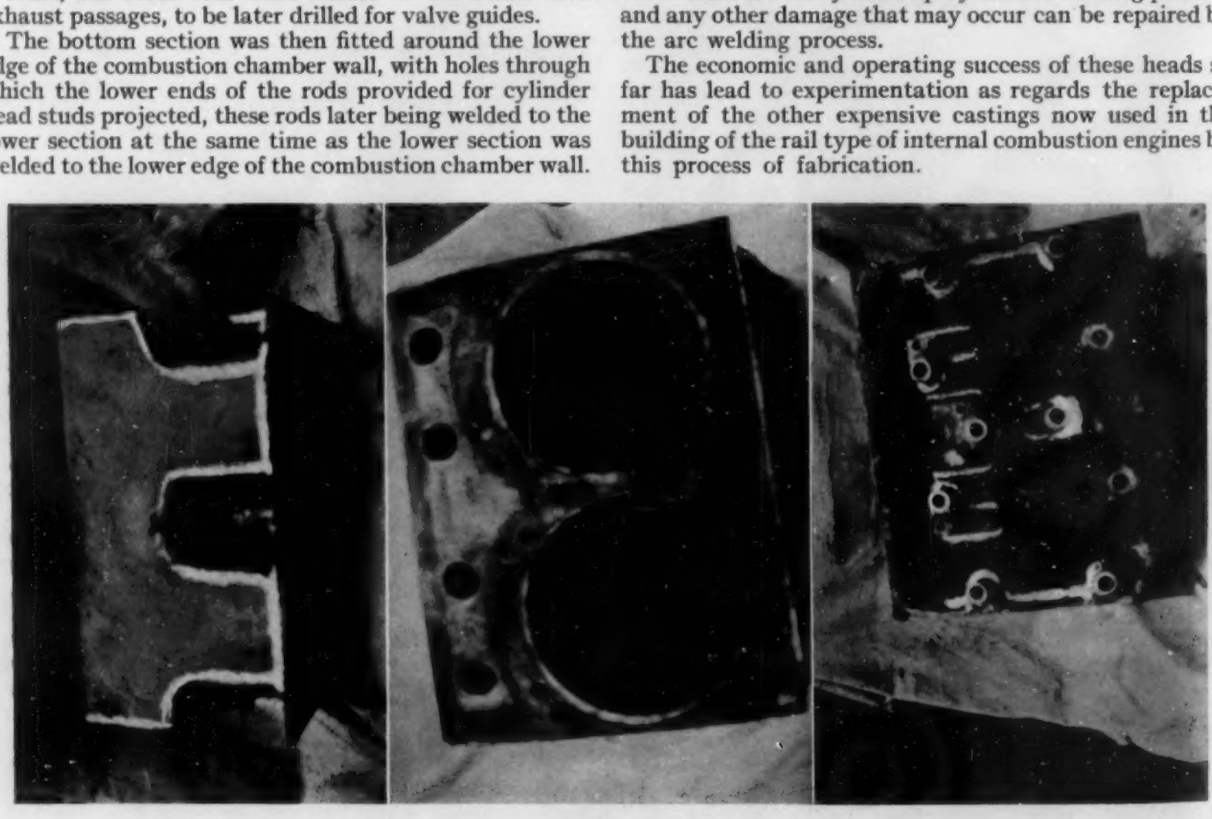
Next, the spark plug rods and valve guide rods are drilled—the hole in the spark plug rods being threaded for the spark plug—the hole in the valve guide rods being machined for the introduction of valve guide bushings.

After this the combustion chamber and valve seats are machined, and such other holes as are required for cooling water pipes, etc., drilled and finished.

These heads, contrary to predictions that had been made, are proving very satisfactory in service and promise long life.

In case of a crack, it can be easily welded again. Worn valve seats are easily built up by the arc welding process, and any other damage that may occur can be repaired by the arc welding process.

The economic and operating success of these heads so far has lead to experimentation as regards the replacement of the other expensive castings now used in the building of the rail type of internal combustion engines by this process of fabrication.



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HOW TO WELD IT

The Design of Jigs and Fixtures for Welding

By **OWEN C. JONES**

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THERE are certain main points that must be considered before setting out to design a jig for any given operation. These can all be summed up as follows:

1. Consider whether the production cost will be sufficiently reduced by the use of a jig. The simplicity or complexity of the jig will have a bearing on this point. The number of articles to be produced will also have to be considered. Having decided that a jig is necessary, the extent of work which the jig must be made to do will also have to be determined. Will it be simply a holding device, or will it have to be more complicated and perform the function of fitting as well? It will very often be found that a fixture or simple holding device will reduce the total cost of production sufficiently to make a saving, whereas the extra cost of fabricating a complicated jig for both holding and fitting might not be worth while. While this is not generally true, it is a factor that must be determined before actual construction is started.

2. Decide upon the position of the locating points. The positioning of these points must be determined both from the point of view of mechanical movement of the jig and also from the point of view of the heat effects both on the jig and on the work. Another item of importance in this connection is that all locating points should be visible to the operator, if it is at all possible to place them so.

3. Consider the speed of the operation. Design the jig so that it will not only work rapidly, but easily. The speed and ease of handling with relation to the operator will have a most decided effect on the output per day.

4. Design the jig so that it is impossible to insert the work out of position. This will make for much quicker and easier assembly and also reduce, if not eliminate, the number of rejects through incorrect assembly.

5. Make some of the locating points adjustable. This will assist in making allowances for rough work from previous operations. It may also make it possible, if necessary, to handle similar work of different size.

6. Clamps, like locating points, should be made to operate rapidly and easily, without depending on great strength on the part of the operator. They should also be positioned so that they are well away from the blowpipe flame and accessible to the operator or helper, whichever will have to insert and remove the work. This last is an extremely important consideration.

7. Make the clamp, if possible, an integral part of the jig. By doing this it will never be mislaid, it will always be in position and, thus, no delay will be experienced in its use.

8. Complicated clamping arrangements should be avoided in every case. The simpler they are, the faster the work can be made ready for welding.

9. In order to avoid springing the work, or the clamp, each clamp should always be positioned opposite a bearing point.

10. The jig or fixture should be made as light as is consistent with the rigidity and stiffness required. By fabricating it whenever possible by the oxyacetylene welding and cutting process, not only can the cost of fabrication be reduced, but also scrap materials can be used in a great many cases. Weight reduction and great strength are also attained. Foundry patterns will seldom be needed and a great deal more versatility in construction will be gained.

11. Avoid sharp corners or edges which might cut or otherwise injure the operator.

12. Handles should be provided for adjusting the various parts of the jig or for convenience in moving the jig from place to place. Conveniences such as these

Fig. 1—A Simple Jig Speeds Production Welding

Fig. 2—Standardization of Product Is a Result of a Carefully Designed Fixture

will greatly speed up the operation and show an increase in daily output.

13. All points of bearing on the jig should be provided with feet. This makes for rigidity in the jig which in turn assures better accuracy and the elimination of warping, bending or any other maladjustment of the work.

14. Sufficient clearance should at all times be provided for the operation of the blowpipe. This is essential as there should be no interference whatsoever with the operator's motions.

15. Do not overlook the fact that the heat from welding flame will cause expansion in metal. Recognize this fact and allow for the control of heat both in the jig and the work.

16. Test the jig carefully before using it by actually using it for a few parts of work especially set aside from the main run of work. Slight deficiencies can then be ironed out before putting the jig into production.

Locating Points

Locating points are important in that they form the best means for positioning the work accurately. A sufficient number of these should be provided to assure correct alignment and, as has been mentioned, they should be so arranged that the possibility of incorrect assembly is eliminated. It is highly important that they be firmly fixed so that they will not be forced out of alignment after the work is clamped. It may also be necessary to have them easily adjustable when the work is heated during the welding.

The kind and number of locating points should be accurately determined, and this is a deciding factor in all jig design, by the nature of the work. They will often be guides by which to line up the outside edges of component parts of the work, or they may consist of holes cut or punched in the work previous to the welding operation, which can be used for aligning the members by means of dowel pins placed through the holes.

Ordinarily, locating points consist of pads, bosses, seats, lugs, pins or studs, vee-blocks, cup and cone arrangements, screws which are either stationary or movable as to location, or the already mentioned finished holes, corners or edges.

Pins or studs that bear on the edges of the work should be flattened to offer a bearing surface rather than simply a point. This will prevent excessive wear. If arranged so they can be removed and replaced they should be firmly fitted to the jig so there will be no movement of the pin in the hole in which it is seated.

Vee-blocks are generally used for aligning cylindrical surfaces. They may be adjustable or stationary. Specially cast blocks, vee-blocks cut from a bar or a billet by the oxyacetylene blowpipe, fabricated from angle iron, channels, tees or sections of old rails, are always useful. Work can be supported in these blocks through their entire length if necessary, or narrow blocks may be placed under critical points.

Cupped sections in the jig may be used to serve the same purpose as vee-blocks. If it is necessary to prevent the rotation of members before or during welding, dowel pins can be used to advantage.

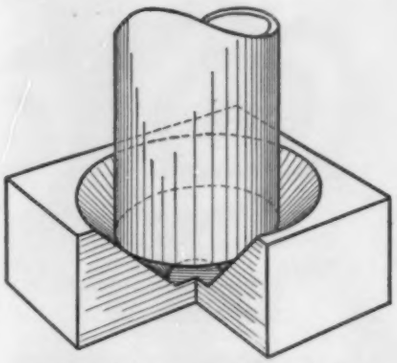
Cup and cone points can be used for locating a smaller cylinder or tube concentric with a larger. These must be carefully placed particularly if the centers of both larger and smaller cylinders must coincide exactly.

If screws points are used for locating points, they should be made long enough so that they project through the walls of the jig. Conical or countersunk points on such screw will be found to give best results. Occasionally it is necessary to make such screws movable sideways or entirely floating so that they can be easily adjusted.

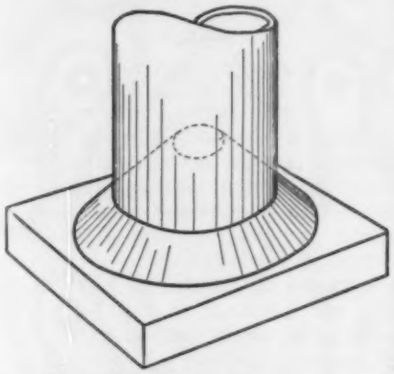
All locating points can be cast solid with a jig body; they can be permanently fixed to it or they may be adjustable. Adjustable stops adapt the jig to irregularities in the work with the use of shims and bushings, or may permit the use of the same jig for more than one size of article. Whatever their use or construction, they must be firm and rigid in whatever position they may be in. Set screws with check nuts are found to be handy as one type of adjustable locating point. The standard, square-headed or headless screws, preferably having a screwdriver slot in the upper end, will be found to be most advantageous. There is one item that is of importance when considering the use of this type of adjustable locating point. Set screws or check nuts should not be used for binding or tightening the work in the jig, but only as methods of locating. They should be so arranged that they will stay in a fixed position once they have been properly adjusted.

The use of finished holes, corners or edges as a means of locating has been mentioned. There are several observations of interest in connection with this type of locating point. As the work progresses from one stage to another during its construction, by the time it is ready for assembly by welding there are usually certain edges which have already been cut to standard size or are already in their correct positions—and the same holds for the holes or corners which can be very conveniently utilized for locating. Obviously, the work that has been carried out previous to the welding operation on a hole or corner or edge that is to be used as a locating point, must be done with sufficient care to assure

Fig. 3—Design the Jig so That the Work Cannot Be Inserted Out of Position



CUTAWAY VIEW SHOWING CUP DEVICE FOR LOCATING CYLINDRICAL OBJECTS



VIEW SHOWING CONE DEVICE FOR LOCATING CYLINDRICAL OBJECTS

Fig. 4—A Cup or Cone Device Is a Splendid Way for Locating Concentric Cylindrical Parts of Work

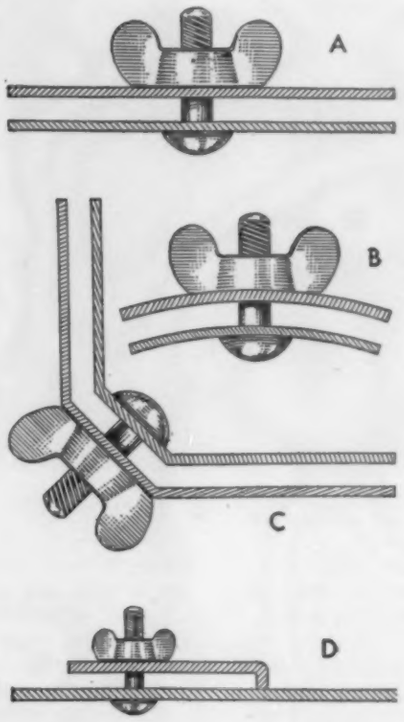


Fig. 5—The H-Clamp Is a Simple and Handy Modification of the Strap

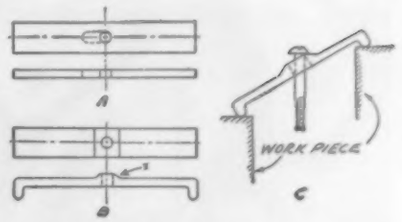


Fig. 6—The Strap Is the Simplest Clamping Device

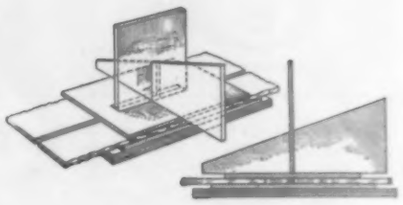


Fig. 7—A Clamping Device Utilizing Both the Strap and the Wedge Principles

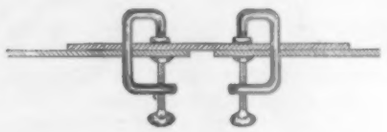


Fig. 8—C-Clamps

accuracy in the welding step of assembly. If the corners, edges or holes are only rough cut in preparation for a further refining step at a later time during the production line, they cannot be used for locating because they will obviously vary in size and this would not permit their use with any assurance of accuracy. Be sure then, if these parts of the work are to be used for locating, that they have already been accurately cut.

Accuracy, therefore, in the preliminary work will mean that the work inserted in the jig for welding will be placed in it with close tolerances. However, the locating points of any jig should have sufficient tolerance to accommodate permissible variations in the preceding operation.

Another point to remember is that the locating points must always be kept at a distance from the weld, if possible, so that the heat of the welding operation will not affect them.

Clamping Devices

Clamping devices on jigs and fixtures may be used either to clamp the work to the jig or the jig to the work. They may consist simply of a loose or movable part which is used to hold the part in place in the jig and which can be swung out of way to facilitate removal or insertion of the work. Often the clamping arrangement makes up the entire jig, while at other times more highly developed specialized types employ clamping appliances that are but attachments to the body of the jig. Whatever the type of grip, however, and in whatever kind of jig it appears, the clamp is a feature of the utmost importance, and the efficiency of the whole operation may depend upon the design of the arrangement by which the work is secured in place.

Simplicity should be aimed for in designing clamps as well as jigs. This means that the clamps must be positive in action, should require a minimum of adjustment and, if possible, be attached to the body of the jig. Clamps should be made as light as possible so that they will require a minimum of effort in operation. Time will then be saved in loading the jig and in removing the finished work after the welding is completed.

When designing clamping devices as few operating screws or handles should be used as will accomplish the purpose, for it takes time to screw even one or two turns per operation. All clamping screws and similar parts should be long enough and so located as to be conveniently taken hold of to operate and they should be of sufficient size to prevent hurting the operator's hand because of pressure necessary to manipulate them. Provide sufficient leverage on the handles so the operator will not be likely to use "persuaders" and break the clamps.

When clamping or locating the work in the jig, it is essential to have the clamping pressure exerted in a direct line against some solid point of support in order to prevent a tilting tendency.

Another important point to be considered with regard to clamping devices is the relation and proximity to the joints to be welded. Here again, as in the case of locating points, careful consideration must be given to the heating effects of the blowpipe flame. The body of the clamp will also expand and contract from the heating and cooling effect and, if too close to the welding position, may become overheated. This must be allowed for and the clamp kept at a sufficient distance, or so designed that it will not be affected by the heat.

Likewise the clamping or locking part of the clamp

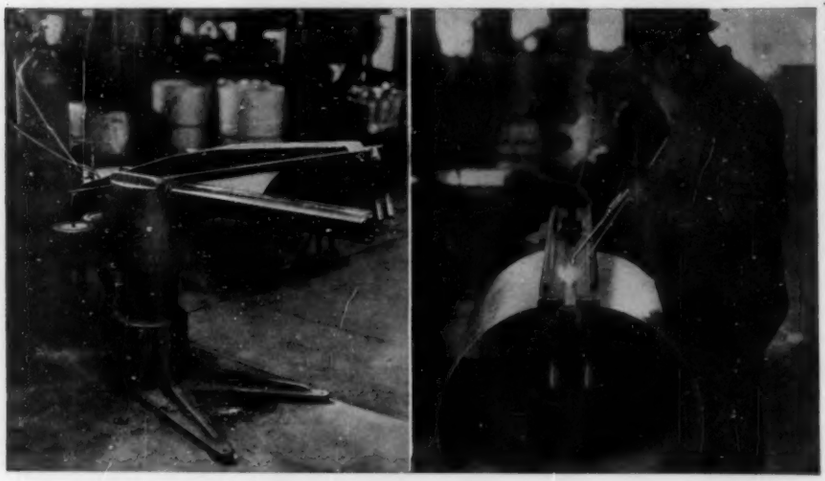


Fig. 9—This Jig Is Designed to Carry Away the Right Amount of Heat from the Weld and Also to Control the Expansion of the Metal

Fig. 10—Vee-Blocks Have Many Uses

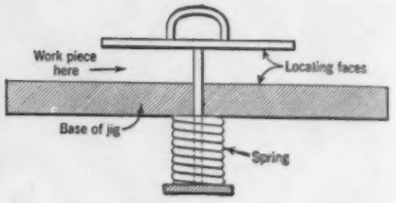


Fig. 11—A Modified Quick-Acting Strap Type Clamp

must be located so that it will not become overheated. Otherwise, time would be lost while the operator waited for the clamp screw or latch to become cool enough for him to release it.

There are four general classes of clamping devices: straps, leaves, wedges or gibs and eccentrics.

Variations of the ordinary strap constitute the simplest type of clamping device. Straps may be merely steel strips of sufficient length to engage in seats in the jig body, or they may be a much more complicated arrangement if necessity calls for it.

The simplest strap consists of a strip of metal with a hole in the center through which a bolt passes to give it pressure. The holes can be either round or elongated to allow for positioning. This is illustrated at *A* in the sketch, Fig. 6. There are obvious deficiencies in this simplest type, the main one being that should the tightening screw have too much pressure upon it, the outer ends of the strap will tend to give outward and lose their holding pressure.

A modified form of this strap which is much more efficient is shown at *B* in the same illustration. *B* is better than *A* because the bearing pressure is only at the ends and even if the strap bends in the middle from the force of the screw, the ends still exert considerable holding pressure. By having a ridge at point *X*, an even pressure is provided from the screw head even if the clamp ends vary in height. An improvement in this can be made by rounding the under points as there will then be an even pressure on the work at any time and in any position of the clamp. The elongated hole is also of value both for adjusting the strap as needed and for removing the work when finished.

The next variation in straps can be considered as those simple straps which are made to bear on more than two points, by having one or both ends of the strap split into as many fingers as necessary.

There are variations to the simple strap clamp known as the H-clamps. Little need be said about these except that they are particularly useful for sheet metal work where great pressure is unnecessary. They can either be flat, curved or modified for special use as needed.

A variation in the design of H-clamps is known as the double L-clamps. It combines, as a matter of fact, features of both the strap and the wedge types. This permits holding of sheets that are resting on a base or where access to the reverse side of the seam is difficult. A section of tee iron about 3 in. in length or a tee made of two pieces of scrap metal welded together is slotted as shown in the drawings, Fig. 7, to receive a wedge-shaped key.

Over the slotted member two washers cut from small pieces of scrap plate are placed. The sheet edges are placed between the jaws formed by the two and the

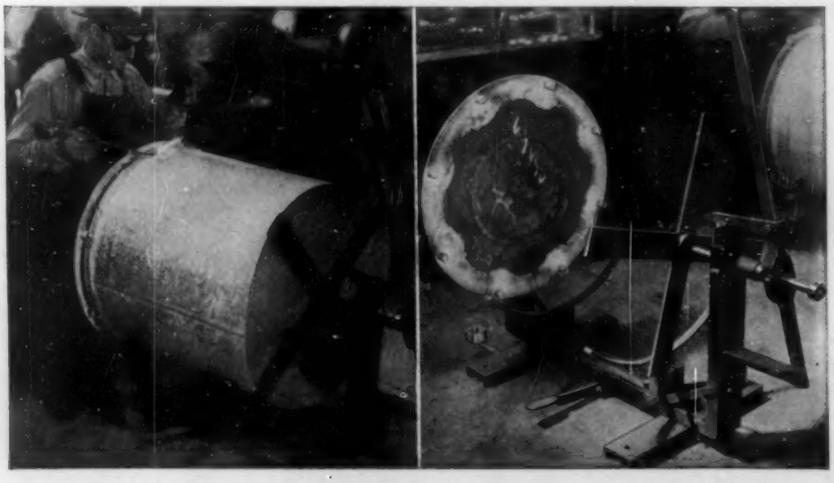


Fig. 12—A Well-Designed Jig. It Is Easily Operated, Holds Parts in Exact Position and Is Convenient to the Operator

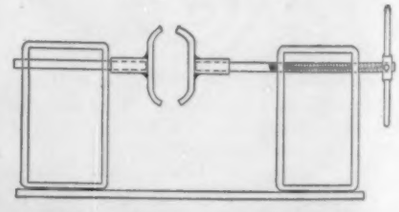


Fig. 13—This Jig for Welding Auto Heaters Employs the Wedge Type Clamping Principle

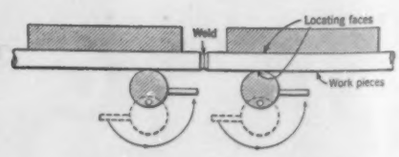


Fig. 14—The Cam and Lever Arrangement Provides Fast and Accurate Clamping

washers are held tight by driving the key into the slot. Such clamps are simple, inexpensive and easily fashioned with the blowpipe from scrap material; they can be made up specially to the requirements of particular jobs.

The next variation of the strap type clamp is that in which the ends are fastened, and the center, by means of an adjusting screw, is used for a bearing point on the work. There are various types of this clamp, varying according to need.

Swinging leaves form a second type of clamp that can be used to advantage. These are particularly designed for simple and extremely rapid insertion or removal of the work. It is unnecessary to say a great deal about this type of holding device, other than that often trigger locks can be used which will do away with the necessity of adjusting a screw. The illustration in Fig. 9 shows variation of this type of clamp. In this one jig the two locking devices on the front end are swinging leaves.

As for wedge or taper gib type clamping devices, there are many forms in use that are practically all variations of the simple wedge principle. The ingenuity of the designer and the specific needs of the work will more or less dictate the necessary deviations in making wedge type clamps to suit the individual needs.

One type of wedge which is used in many cases is illustrated in the sketch in Fig. 13. It is composed of two partially vee-sided sections which oppose each other on opposite sides of the work. They can be arranged in various ways to hold cylinders or other round sections in a firm position. Occasionally a screw device is used for pressure on one side while the other wedge is fixed in position.

Eccentric clamping arrangements are also found to be a great value in holding devices, particularly because of the ease with which the work may be inserted and removed without the necessity of screwing down bolts for tightening purposes. There are two types of eccentric clamping arrangements generally in use, the eccentric shaft and the eccentric lever. Both of these are of greater use for locking devices than for any other purpose.

The eccentric shaft, for instance, is so arranged that when it is given a half turn, the wider portion of the shaft bears down on a connecting rod which exerts the pressure. This arrangement will only give about $\frac{1}{8}$ to $\frac{1}{4}$ of an in. throw and can, therefore, only be used for a fine adjustment.

The eccentric lever is another arrangement which is used only for fine work, and should not be used in rough work. Here again the throw is small. The illustration in Fig. 14 shows to best advantage this principle.

The Control of Heat Effects

The control of heating effects from the blowpipe flame is a matter which is of the utmost importance in the design of jigs and fixtures for welding. Primarily it must always be remembered that heated metal expands and cooling metal contracts. To compensate for this effect on the part being welded a jig or fixture, particularly when complicated, needs to be most carefully designed with this fact in view. Do not lose sight of the fact that parts of the jig itself also expand when they become heated. Design the jig so that it will not become heated or, if it does, so that locating points and other parts can be adjusted correspondingly.

There are generally four ways of handling the expansion problem:

(1) Expansion can be controlled in part; (2) it can be completely resisted in light work permitting the metal in the work to take up the resulting strains; (3) the jig can be designed to carry away the right amount of heat so that there will not be any noticeable effect in the work itself; (4) the welding operation itself can be controlled so that a minimum of heat will be allowed to pass into the work. It must be remembered that these four divisions of control overlap one another greatly, so that a part that carries the heat away from the work may also exert the controlling action on the metal itself.

There are various devices in use controlling the action of work when it undergoes expansion and contraction from the heat of the welding flame. Of these wedges, C-clamps and the use of heavy objects which by mere weight will have a certain effect are the most commonly used.

Wedges are used, for instance, on the welding of large vessels. Here a longitudinal seam of considerable length is spaced out the correct distance by means of wedges which, as the welding progresses, can be cut out or removed and pushed further along. C-clamps, strap type clamps and H-clamps, for lighter work, also serve the same purpose and the action of the clamp depends entirely upon the amount of pressure exerted by the screw part of the clamp.

In very light sheet metal work, the expansion and contraction can be completely resisted, or made to expend its effort in the welded seam itself. This obviously requires a heavy and strong holding device which by its very nature permits so little play in the metal that all of the

Fig. 15—Exact Vertical Alignment Is Assured in This Fixture for the Assembly of Aluminum Chairs

contraction and expansion would have to be taken up in the weld. Aluminum welding, for instance, where the welding rod is so made as to solidify at a lower temperature than the base metal, makes use of this principle. Actually, complete resistance to the action of the heated metal is practically impossible, particularly in heavier work, and is not, except in a few cases, the best method of minimizing distortion. Occasionally a jig can be designed to carry away sufficient heat and at the same time resist the expanding action so that little distortion takes place. Such a jig would have to be exceedingly strong and rigid, possibly made of cast metal, and either water-cooled or otherwise of such a nature that the heat is absorbed rapidly.

This would almost be a border-line case combining the principle of carrying away the right amount of heat from the work and of resisting the effects of the heat.

Heavy bars, strips of heavy plate or other bulky sections of iron or steel can be located along the joint and can be so placed that most of the heat is picked up and not permitted to pass on into the work except just adjacent to the weld. When short seams in light sheet are welded in considerable numbers, special equipment to regulate expansion and contraction is justified. This ordinarily takes the form of specially cast members which may be hollow and also water-cooled.

One of the best ways of controlling the heat from welding operations is by the regulation of the oxyacetylene flame itself. By the use of the smallest possible flame consistent with doing good work, by using a welding rod with a lower melting temperature than the base metal, such as bronze on steel, or by employing a technique which can be carried out rapidly, may serve to largely eliminate distortion. This again is a matter that must be worked out by the individual requirements of the job.

Operation and Regulation of Production

Plan the method of handling the welding operation so the value of the jig will be realized to the highest degree. In production work, either fabrication or repair, where a jig forms the basis of handling the work, this question is of highest importance, because the jig, unless operated properly, will not give the best results. A convenient position for the operator is essential. Experience has shown that the best results are obtained when the welding is done at about the level of the operator's waist. This would indicate that the jig must be placed on a suitable table, or otherwise situated at the correct height. If the jig is to be loaded and unloaded by a helper it should be accessible from both sides, leaving the operator undisturbed on one side. It may be necessary to place the helper next to a conveyor line or trucking aisle from which the parts are delivered and completed parts removed.

The use of helpers is another point to be considered. The management should decide whether a helper should be employed to load and unload the jig, or whether the operator should do this work himself. Here the time element is important. If the loading and unloading operations consume only a small amount of time as compared with the welding operation, it is obvious that the helper is not essential. On the other hand, if the welding operation takes but a few seconds, then a helper is of the greatest value because he can be setting up the work and removing it, leaving the operator undisturbed at the welding.

Another point to remember is that the operator, particularly when working on heavy material, will need relaxation from one position. He can be relieved from this work by another, or by changing position. These facts must be considered and accounted for before starting on a design.

Occasionally it is possible for the same crew to operate two jigs, the helper unloading and reloading one as the welder works on the other jig. Here generally the operator is located between the two jigs, so he can reach both conveniently.

Sometimes the positions of welder and helper are made interchangeable, and two individuals, both able to make good welds, relieve each other at definite intervals.

Another factor that should be considered is whether a semi-automatic jig is indicated. This is a more complicated problem. It is one which should be entirely determined by a comparison of estimated costs of the work done by means of a complicated jig as against the cost of the work when using a simple, hand-operated jig. When a semi-automatic jig is indicated, it should be so arranged that the operator has before him and at all times the point that is next to be welded. Turning may be done by the operator's left hand, by a motor or by a helper.

Pedals for ratchets or releasing mechanism should obviously be placed at the point where they will be most convenient for the operator. If the jig is to be operated by a helper the mechanism should be located in such a position that it will not interfere with the welder.

Speed regulating devices on motor-operated jigs should preferably be placed where the welder can manage them with his foot. Often it is possible to make the jig bodies detachable from the rotating apparatus so one jig can be unloaded and reloaded while another is before the welder; the change from one piece of work to another is then made with a minimum of interruption to the operator.

CURRENT WELDING LITERATURE

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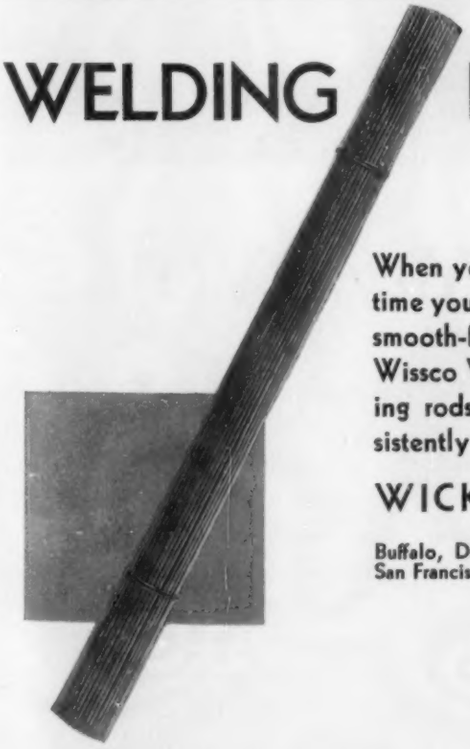

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